# CONCRETE AND CONSTRUCTIONAL ENGINEERING



INCLUDING PRESTRESSED CONCRETE

FEBRUARY 1958 VOL. LIII. NO. 2

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## LEADING CONTENTS



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CONCRETE
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Volume LIII, No. 2. LONDON, FEBRUARY, 1958.

# EDITORIAL NOTES

## Standards of Knowledge.

THE stipulation that applicants for engineering posts must be university graduates or members of a professional institution seems to have originated about thirty years ago with some Government departments and local authorities. It is now common in the nationalised industries, and an increasing number of private employers are making similar stipulations. If, however, an employer accepts these degrees and memberships as a substitute for his own assessment of the suitability of an applicant it seems that he may be very much misled. This is shown in a recent report of the Civil Service Commissioners, in which it is stated, in connection with applicants for engineering and technical posts : " There was no lack of applicants for most of the posts advertised, but their general quality was poor. We were concerned to notice how many candidates fell short of the standard of basic technical knowledge implied by the qualifications they held. . . . Many of those recently qualified seemed to have retained a disturbingly small store of lasting knowledge, and this of a somewhat superficial character." In the view of the Commissioners, the possession of a degree, or membership of a professional institution, is not always a reliable indication of the suitability of an applicant. The more engineering appointments become " closed shops " to members of professional institutions, and the more these memberships are accepted as a criterion, the more important it is that they should represent a certain, and known, standard of knowledge and experience, and, if alternative qualifications are mentioned, that these should represent the same standard.

A report issued by the Institution of Civil Engineers states that of 501 candidates who attended parts I and II of the examination for Associate Membership in October 1956 only 216 were successful. No results are given for the Professional Interview but, by comparing the number of candidates who attended the Interview with the number elected to Associate Membership over a number of years, it appears that at least 80 per cent. are successful. The training necessary to become an Associate Member may be obtained in several ways, and it is interesting to consider the results of different methods of study. It is likely that of those who start a course of study with the intention of presenting themselves for the examination, about one in ten may eventually become Associate Members ; of those who commence a course of studies leading to a Higher National Certificate or equivalent qualification, about four of every ten may become Associate Members ; and of

internal students at a university about three out of four may become Associate Members. Such results are no doubt a measure of the difficulties, and consequently the advantages and disadvantages, of the three methods of study, and it is clearly advantageous to be an internal student at a university, as the Council has pointed out on several occasions. It is by no means certain, however, that the standards set by university examiners are equivalent to those set by the Institution's examiners. The percentage of successful candidates at the Institution's examinations is less than half of that at university examinations. It seems unlikely that this large difference can be accounted for by differences in the quality of the teaching, nor is there likely to be so large a difference in the quality of the candidates. University students are often examined by their lecturers, who have a knowledge of their capabilities which is not solely based on the evidence offered by the examination papers; in many ways this is a desirable feature of university training, but it is possible that some graduates who are now exempted from parts I and II of the Institution's examination might find difficulty in passing them. It is not suggested that the Institution should lower its own examination standards, but it may be that the present policy of granting full exemption from parts I and II of the Institution's examination to all graduates is not the best that could be devised; some form of examination for such graduates, in addition to the Professional Interview, might help to ensure that all Associate Members are known by the Institution to have a certain standard of knowledge and skill.

Under the present system a university graduate is required to spend at least one year in a drawing office and one year on a construction site before presenting himself for the Professional Interview; this usually lasts for less than half an hour, and the candidate is then required to write an essay on one subject set by the interviewers. These are men of much professional experience, who devote great care and skill to their task, but it is doubtful whether this is the best test of engineering skill and knowledge; it seems hardly possible that any candidate can obtain sufficient experience of the art of engineering (as distinct from the science of applied mathematics) in two years to qualify himself to be a Chartered Civil Engineer. Candidates who qualify by means of part-time study are required to have spent at least five years in practice. This is much more desirable, but it is not clear why the same requirement does not apply to university graduates. A candidate who has attended a course of part-time study during the time that he has been in employment is likely to have a more balanced knowledge of both the theory and practice of engineering than one whose practical training consists only of two years, yet the present system tends to discourage part-time training.

It would perhaps be better if all candidates were examined in such a way as to test their knowledge of engineering rather than their ability to memorise applied mathematics, so that the candidates with the best knowledge of engineering have the best chance of election. It would be interesting to know what proportion of the unsatisfactory applicants referred to by the Civil Service Commissioners had been exempted from parts I and II of the examination of the Institution of Civil Engineers. The problem would be of little importance if a student's prospects of advancement in his chosen profession did not often depend upon his election to membership of the Institution and if it were not generally assumed by employers that newly-elected members of the Institution necessarily have the same knowledge and experience.

# A Precise Method of Moment Distribution.

By A. E. HOLDAWAY, B.Sc., Ph.D., A.M.I.Struct.E.

WHEN a continuous beam is subjected to a number of different loading conditions its analysis by the method of moment distribution is laborious, and much time can be saved if a procedure is used which avoids the arithmetical work of distribution. The method here described requires the preliminary calculation of a few simple values, after which the bending moments caused by any loading may be obtained in a single cycle. The method is applicable to both prismatic and non-prismatic beams, or to a combination of both. Except for the stiffness factors and carry-over factors required for non-prismatic beams, which are obtainable from published tables or graphs, or may be calculated in the usual manner, no tables, graphs, or charts are required.

DERIVATION OF FORMULÆ.—The slope deflection equations which are applicable to a prismatic or non-prismatic beam are:

$$M_{AB} = \frac{EI_0}{l}(K_1\theta_A + K_2\theta_B) \quad . \quad . \quad . \quad . \quad (1)$$

$$M_{BA} = \frac{EI_0}{l}(K_2\theta_A + K_3\theta_B) \quad . \quad . \quad . \quad . \quad (2)$$

in which $l$, $\theta_A$ and $\theta_B$ have their usual significance; $I_0$ is the value of $I$ for some reference section; $K_1$ is the stiffness factor at A; $K_3$ is the stiffness factor at B; $\frac{K_2}{K_1}$ is the carry-over factor from A to B $= r_{AB}$; $\frac{K_2}{K_3}$ is the carry-over factor from B to A $= r_{BA}$.

From (1) and (2) $\quad \dfrac{M_{BA}}{M_{AB}} = \dfrac{K_2\theta_A + K_3\theta_B}{K_1\theta_A + K_2\theta_B} = \phi.$

Therefore $\quad K_2\theta_A + K_3\theta_B = K_1\phi\theta_A + K_2\phi\theta_B,$

from which

$$\theta_A = \frac{\theta_B(K_2\phi - K_3)}{K_2 - K_1\phi} \quad . \quad . \quad . \quad . \quad . \quad (3)$$

and

$$\theta_B = \frac{\theta_A(K_2 - K_1\phi)}{K_2\phi - K_3} \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Substituting (3) in (1), $\quad M_{AB} = \dfrac{EI_0}{l}\left[K_1\theta_A + \theta_A\dfrac{K_2(K_2 - K_1\phi)}{K_2\phi - K_3}\right].$

RESTRAINT FACTOR.—The restraint factor at A is denoted by

$$R_{AB} = \frac{M_{AB}}{\theta_A} = \frac{EI_0}{l}\left[K_1 + K_2\frac{(K_2 - K_1\phi)}{K_2\phi - K_3}\right] = \frac{EI_0}{l}\left(\frac{K_2^2 - K_1K_3}{K_2\phi - K_3}\right)$$

$$= \frac{EI_0}{l}\left[\frac{1 - \frac{K_2}{K_1}\cdot\frac{K_2}{K_3}}{\frac{1}{K_1}\left(1 - \frac{K_2\phi}{K_3}\right)}\right] = \frac{EI_0K_1}{l}\left(\frac{1 - r_{AB}r_{BA}}{1 - r_{BA}\phi}\right) \quad . \quad . \quad (5)$$

This is a general formula for prismatic and non-prismatic beams.

For prismatic beams, $\dfrac{I_0K_1}{l} = K$, $r_{AB} = r_{BA} = 0{\cdot}5$,

and
$$R_{AB} = \frac{EK\left(1 - \frac{1}{4}\right)}{1 - \frac{\phi}{2}} = \frac{1{\cdot}5EK}{2 - \phi} \qquad (6)$$

Similarly, by substituting (4) in (2),
$$R_{BA} = \frac{M_{BA}}{\theta_B} = \frac{EI_0K_3\phi}{l}\left(\frac{1 - r_{AB}r_{BA}}{\phi - r_{BA}}\right) \qquad (7)$$

and for prismatic beams,
$$R_{BA} = \frac{1{\cdot}5EK}{2 - \frac{1}{\phi}} \qquad (8)$$

These restraint factors were first derived by Professor Hardy Cross, using the method of moment distribution. The derivation by the method of slope deflection shows the close connection between the two methods.

CONTINUITY FORMULA.—Proceeding from the left-hand end of the beam towards the right, the values of $\phi$ are interrelated. The first is determined from the known, or assumed, end conditions (free, fixed, or some intermediate degree of fixity) at the outer support. From this first value the next value of $\phi$ in the second span can be found, in the case of a prismatic beam, by a simple continuity formula derived below, and thus all successive values of $\phi$ and $R$ can be easily calculated.

In the case of non-prismatic beams it is not easy to establish a general continuity formula, and successive values of $\phi$ should be determined by inserting the known values of $I_0^{AB}$, $K_3$, $r_{AB}$, $r_{BA}$, and $l_1$, in the equation for the first span, and $I_0^{BC}$, $K_1$, $r_{BC}$, $r_{CB}$, and $l_2$ in the equation for the next span. In the first span $\phi_1$ is known from the end condition, and hence $\phi_2$ is determinate. The value of $\phi$ being known, the restraint factors can be computed.

An equation can be derived for this purpose from the fact that $\Sigma M$ at each support $= 0$. From *Fig.* 1, the general equation is
$$\frac{-I_0^{AB}K_3(1 - r_{AB}r_{BA})}{l_1(1 - r_{BA}\phi_1)} = \frac{I_0^{BC}K_1\phi_2(1 - r_{BC}r_{CB})}{l_2(\phi_2 - r_{CB})} \qquad (9)$$

In the case of non-prismatic beams, $I_0^{AB}$, $I_0^{BC}$ ($= I$ at some particular section, generally the minimum section) and $l_1$ and $l_2$ are calculated for the actual beam, and the carry-over factors $r$ are obtained from published tables or graphs, or

| Joint: | A | $l_1$ | B | | $l_2$ | C |
|---|---|---|---|---|---|---|
| True Stiffness K: | | | $K_3$ | $K_1$ | | |
| Modified Stiffness R: | | $\dfrac{-I_0^{AB}K_3(1-r_{AB}r_{BA})}{l_1(1-r_{BA}\phi_1)}$ | | $\dfrac{+I_0^{BC}K_1\phi_2(1-r_{BC}r_{CB})}{l_2(\phi_2-r_{CB})}$ | | |

Fig. 1.

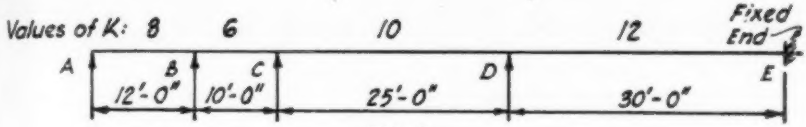



Fig. 2.

| Values of K: | 8 | 6 | 10 | 12 |
|---|---|---|---|---|
| Joint: | A — B | B — C | C — D | D — E |
| $\phi \longrightarrow$ | 0 | 0·286 | 0·206 | |
| $\longleftarrow \phi$ | | 0·332 | 0·308 | 0·500 |
| $R = K/2 - \phi$: | 4·00 | 3·60   3·50 | 5·92   5·58 | 8·00 |
| Distⁿ Coeff: | 0·53 | 0·47   0·37 | 0·63   0·41 | 0·59 |

Fig. 3.

may be calculated directly. For prismatic beams

$$\frac{I_0^{AB}K_3}{l_1} = \frac{I_0^{BC}K_1}{l_2} = K, \text{ and all carry-over factors are } 0{\cdot}5.$$

Therefore

$$\frac{-K_3\left(\frac{3}{4}\right)}{1 - \frac{\phi_1}{2}} = \frac{K_1\phi_2\left(\frac{3}{4}\right)}{\phi_2 - \frac{1}{2}},$$

from which

$$\phi_2 = \frac{1}{2 + \frac{K_1}{K_3}(2 - \phi_1)} \quad . \quad . \quad . \quad . \quad (10)$$

The values of $\phi$ and $R$ are the only constants required in this procedure and for prismatic sections the formulæ are simple and easy to remember.

## Examples.

EXAMPLE I.—In the four-span prismatic continuous beam shown in *Fig.* 2 the end A is assumed to be hinged and end E fully fixed. The constants in *Fig.* 3 are obtained as follows.

Values of $\phi$.—

(a) A to E: In span AB, $\frac{M_{AB}}{M_{BA}} = 0$; therefore $\phi_1 = 0$.

In span BC, $\phi_2 = \frac{1}{2 + \frac{K_2}{K_1}(2 - \phi_1)} = \frac{1}{2 + \frac{6}{8}(2 - 0)} = 0{\cdot}286.$

In span CD, $\phi_3 = \frac{1}{2 + \frac{10}{6}(2 - 0{\cdot}286)} = \frac{1}{2 + 2{\cdot}86} = 0{\cdot}206.$

No value is required at E.

(b) E to A: In span DE, $\dfrac{M_{ED}}{M_{DE}} = 0{\cdot}5$; therefore $\phi = 0{\cdot}5$.

In span CD, $$\phi_2 = \frac{1}{2 + \dfrac{K_2}{K_1}(2 - \phi_1)} = \frac{1}{2 + \dfrac{10}{12}(2 - 0{\cdot}5)} = 0{\cdot}308.$$

In span BC, $$\phi_3 = \frac{1}{2 + \dfrac{6}{10}(2 - 0{\cdot}308)} = 0{\cdot}332.$$

Values of *R*.—

Since relative values only are required, the constant $1{\cdot}5E$ which occurs in the numerators of the formulæ for *R* may be omitted.

At support B,

$$R_{BA} = \frac{K}{2 - \phi} = \frac{8}{2 - 0} = 4, \text{ and } R_{BC} = \frac{K}{2 - \phi} = \frac{6}{2 - 0{\cdot}332} = 3{\cdot}60.$$

At support C,

$$R_{CB} = \frac{6}{2 - 0{\cdot}286} = 3{\cdot}50, \text{ and } R_{CD} = \frac{10}{2 - 0{\cdot}308} = 5{\cdot}92.$$

At support D,

$$R_{DC} = \frac{10}{2 - 0{\cdot}206} = 5{\cdot}58, \text{ and } R_{DE} = \frac{12}{2 - 0{\cdot}5} = 8{\cdot}00.$$

These constants enable the bending moments due to any loading on the beam to be distributed directly. For example, the distribution of a bending moment of + 10 ft.-tons at support B is shown in *Fig.* 4.

The final moments are obtained by direct distribution in a single cycle in line (*a*). The moment at B is distributed in accordance with the factors given in *Fig.* 3. The moment at C is − 4·7 multiplied by the values of $\phi$ for BC,

| Joint. | A | B | B | C | C | D | D | E |
|---|---|---|---|---|---|---|---|---|
| (a) | - | +10<br>-5·3 | -4·7 | -1·56 | +1·56 | +0·48 | -0·48 | -0·24 |
| Final Moment | - | +4·7 | -4·7 | -1·56 | +1·56 | +0·48 | -0·48 | -0·24 |

**Fig. 4.**

| Joint: | A | B | B | C | C | D | D | E |
|---|---|---|---|---|---|---|---|---|
| F.E.M.: | | +16 | -40 | +20 | -10 | +80 | -60 | +40 |

**Fig. 5.**

| Joint: | A | B | B | C | C | D | D | E |
|---|---|---|---|---|---|---|---|---|
| F.E.M. | | +16 | −40 | +20 | −10 | +80 | −60 | +40 |
| B: | 0 ← | +12·7 | +11·3 → | +3·75 | −3·75 → | −1·15 | +1·15 → | +0·58 |
| C: | 0 ← | +1·1 | −1·1 ← | −3·7 | −6·3 → | −1·95 | +1·95 → | +0·97 |
| D: | 0 ← | −0·49 | +0·49 ← | +1·69 | −1·69 ← | −8·2 | −11·8 → | −5·9 |
| Σ Final Mts.: | 0 | +29·31 | −29·31 | +21·74 | −21·74 | +68·70 | −68·70 | +35·65 |

**Fig. 6.**

| Values of K: | 8 | | 6 | | 10 | | 12 | |
|---|---|---|---|---|---|---|---|---|
| Joint: | A | B | B | C | C | D | D | E |
| $\phi$ → | ∞ | 0 | 0·286 | | | | | |
| ← $\phi$ | | | | | | | 0·5 | 0·2 |
| R: | 0 | 4·00 | | | | | 8·00 | ∞ |

**Fig. 7.**

giving $-4·7 \times 0·332 = -1·56$ ft.-tons. Similarly, the moment at D is $+1·56 \times 0·308 = +0·48$ ft.-ton and the moment at E is $-0·48 \times 0·5 = -0·24$ ft.-ton. These moments are the final distributed moments at the supports. The moment at A is $+4·7 \times 0 = 0$.

EXAMPLE II.—The fixed-end moments are assumed to be those given in *Fig.* 5. The values of $R$ are unchanged.

The distribution is shown in *Fig.* 6. Line ($B$), for example, gives the distribution of the unbalanced moment at joint B using the calculated distribution factors. The moment is $-24$ ft.-tons, and is balanced by a moment of $+12·7$ ft.-tons at BA and $+11·3$ ft.-tons at B. The moment of $+12·7$ gives rise to carry-over moments to the left of B giving $+12·7 \times 0 = 0$ ft.-ton at A. The moment of $+11·3$ ft.-tons causes carry-over moments to the right of B, giving $+11·3 \times 0·332 = +3·75$ ft.-tons at C, $-3·75 \times 0·308 = -1·15$ ft.-tons at D, and $+1·15 \times 0·5 = +0·58$ ft.-ton at E. The unbalanced moments at other joints are carried over to left and right in a similar manner, and the final moments are the sum of the fixed-end moments and the moments carried over at the supports. The arrows indicate the direction of carry-over.

This method of tabulating the values of $\phi$ and $R$ is useful in practice, but is not mathematically correct as it does not give the true stiffnesses at the ends of the beam. At A the true stiffness is 0, and at B it is $\infty$. According to the present tabulation the stiffness at A would be $\dfrac{8}{2 - 0·23} = 4·52$. This, however, is unimportant since the stiffnesses at the ends are not used in distribution. All internal stiffnesses are correct, and these are the ones used in distribution. The advantage of this method of tabulation is that the values of $R$ and $\phi$ are more easily calculated and that values of $R$ are found from the formula $R = \dfrac{1}{2 - \phi}$ only. The following method of tabulation could be used if desired (see *Fig.* 7).

At A, $\frac{M_{BA}}{M_{AB}} = \infty$. Therefore $R_{AB} = \frac{1}{2 - \phi_1} = 0$.

At B; $R_{BA} = \frac{K}{2 - \frac{1}{\phi_1}} = \frac{8}{2 - \frac{1}{\infty}} = 4$.

Also $\frac{M_{AB}}{M_{BA}} = 0$. Hence $\phi_2 = \frac{1}{2 + \frac{6}{8}(2 - 0)} = 0{\cdot}286$.

At the fixed end E, $\frac{M_{DE}}{M_{ED}} = 2$. Therefore $R_{ED} = \frac{1}{2 - 2} = \infty$.

At D, $\frac{M_{ED}}{M_{DE}} = 0{\cdot}5$. Therefore $R_{DE} = \frac{12}{2 - 0{\cdot}5} = 8$.

These results are identical with those first obtained.

## Lectures on Building.

The following lectures have been arranged by the Ministry of Works. Admission is free.

Introduction to Prestressed Concrete, by R. C. Blyth. County Library, William Street, Slough. February 18. 7.15 p.m.

Pre-Planning: Programming and Progressing, by G. J. J. Hunt. Technical College, Stoke Park, Guildford. February 19. 7.15 p.m. And Emmanuel Church Hall, Okehampton Road, Exeter. February 25. 7 p.m.

Powered Hand Tools, by A. F. Coare. Technical College, Park Road, Mexborough. February 20. 7.15 p.m.

Work Study in the Building Industry, by K. C. Symons. Arts School, St. George Street, Norwich. February 26. 7.30 p.m.

Concrete Placing and Formwork, by A. B. Harman. Technical College, King's Road, Reading. February 26. 7.15 p.m.

Weathering and Deterioration of Concrete and Cement Renderings, by C. Hobbs. Institute of Technology, Great Horton Road, Bradford. February 27. 7.15 p.m.

Soil Mechanics in the Building Industry, by D. J. Harris. School of Building, Lime Grove, London, W.12. February 27. 7 p.m.

Practical Formwork Design and Construction for Concrete, by J. G. Richardson. S.W. Essex Technical College, Forest Road, London, E.17. February 14. 7.15 p.m.

Thermal Insulation of Buildings, by J. Lawrie. Technical College, Reigate Road, Ewell. February 19. 7.15 p.m. Also by F. King. Technical College, Chesterfield Road, Mansfield. February 27. 7.15 p.m.

Thermal Insulation, by J. A. Godfrey. Gas Showrooms, Osborne Street, Grimsby. February 19. 7.15 p.m.

## Laying Concrete Roads without Fixed Forms.

About twelve miles of concrete road, with two carriageways each 24 ft. wide and 8 in. thick, have been laid in Colorado, U.S.A., without fixed forms along the sides of the slabs. The road was laid with a paving machine comprising spreaders, vibrators, and smoothing devices, carried by two crawler tracks travelling on the prepared foundation outside the edges of the slab. Fixed to the sides of the machine were "slip-forms", that is forms that moved with the machine and formed the sides of the slab. These forms extended 57 ft. to the rear of the machine, and the extensions were kept the required distance apart by light lattice girders spanning over the slab. The machine travelled at the rate of 3 ft. 6 in. a minute, and by the time the ends of the forms had slid away from the slab the concrete was hard enough for the edges to retain their shape without support. It is stated that the result was satisfactory, and that there was a saving of 8 per cent. compared with the use of fixed road forms. The machine was driven by two 35-h.p. petrol engines.

# Structures for the Brussels Exhibition.

MANY of the structures now in course of erection for the Universal Exhibition, to be held in Brussels from April to October 1958, are of concrete. Most of the buildings are to be removed after the exhibition, but one of the few permanent buildings will be the "Atomium", which will be a representation of an alpha iron crystal magnified 200,000,000,000 times.

The height of the structure will be about 360 ft. *Fig.* 1 shows a model of the completed structure, and *Fig.* 2 shows the structure in course of construction. The foundations consist of four groups of piles with reinforced pile caps 10 ft. deep. Each pile is 57 ft. long, with a safe bearing capacity of 55 tons.

Each of the eighteen connecting tubes will exceed 100 ft. in length and will weigh 27 tons; escalators will be housed in six of them. The steel frames which support the outer spheres weigh 100 tons each. The nine spheres will be of polished aluminium supported by a steel frame and will be about 60 ft. diameter. Each sphere will have two floors which will be used as exhibition halls, and the top-most sphere will contain a restaurant. They will be lighted by electricity, and will also have portholes of plexiglass. The central mast will contain a lift for 20 passengers, and is expected to carry 400 to 500 visitors per hour. The whole construction will weigh 2500 tons and will contain 2000 tons of steel.

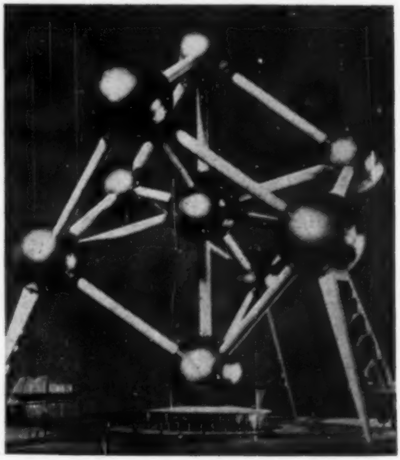


**Fig. 1.—The "Atomium".**

**Fig. 2.—The "Atomium" during Construction.**

The British pavilion (*Fig.* 3) has no natural lighting in the interior. It was designed by Mr. Howard V. Lobb and Mr. John Ratcliffe, and is in the form of three connected crystals which culminate in spires forming the roof. The walls are decorated with coloured "eyelets" which, although they admit little light from the outside, give an impression of stained glass windows; the ceiling is 42 ft. high.

The dominating structure of the Belgian civil engineering section is a reinforced concrete cantilever 240 ft. long, from which is suspended a reinforced concrete walkway (*Fig.* 4).

The permissible stresses assumed in the calculations for the cantilever were 1422 lb. per square inch compression in the concrete, 19,910 lb. per square inch tension in the mild steel reinforcement, and 29,870 lb. per square inch tension in the deformed steel reinforcement. The cross section of the cantilever, the shape of which forms an inverted A, varies in such a way that the compressive stress in the concrete is constant throughout the length.

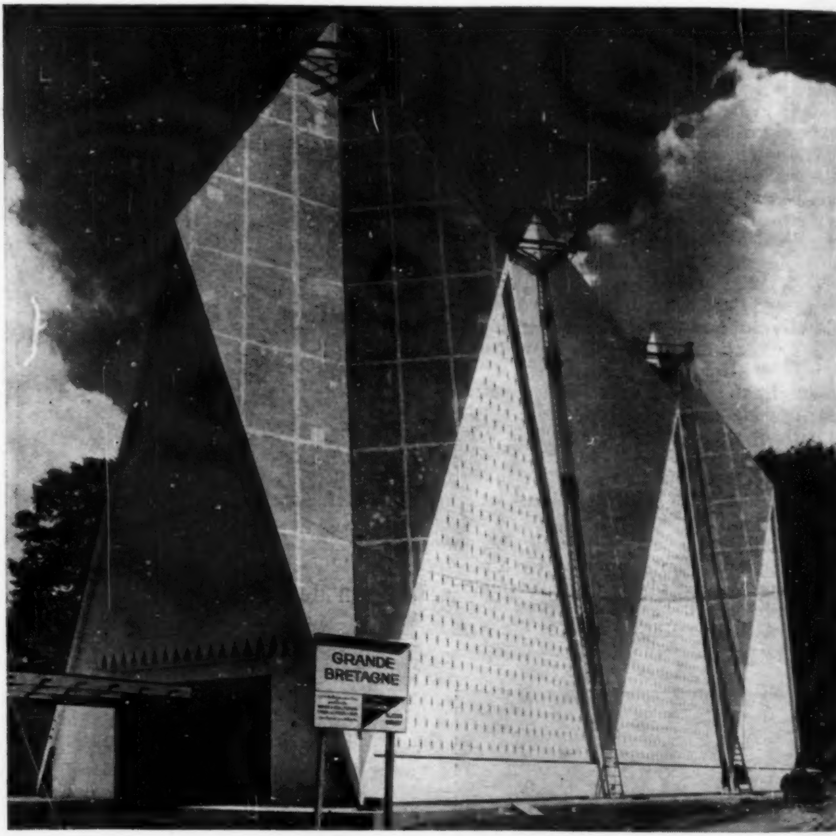



**Fig. 3.—The British Pavilion.**

**Fig. 4.—A Cantilever 240 ft. Long.**

The walls of the cantilever are stiffened by ribs and a transverse slab. The thickness of the concrete varies from 1·6 in. to 4·8 in. The reinforcement at the root of the cantilever consists of fifty-four bars of 1·3 in. diameter in each side wall, and thirty similar bars in the intermediate horizontal slab. *Figs.* 5 and 6 show the structure in course of construction.

The weight of the structure is transmitted to the ground by a single pillar of variable section, which becomes triangular at ground level; the base of the triangle is about 10 ft. long and its height is about 5 ft.; the load on the pillar is about 1200 tons. Two side struts are used to ensure the transverse stability of the structure; their thickness varies from 2·4 in. at the top to 40 in. at the base, where the cross-sectional area is 2080 sq. in. The maximum load on each strut is about 725 tons.

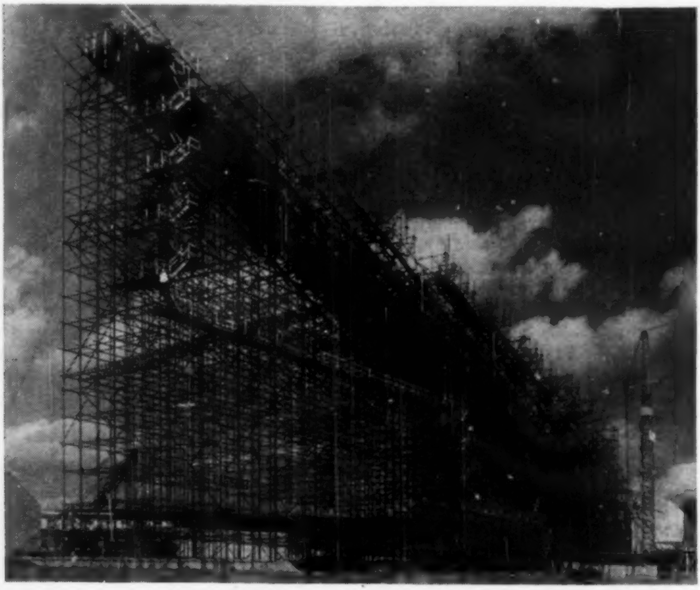


**Fig. 5.—Belgian Civil Engineering Building during Construction.**

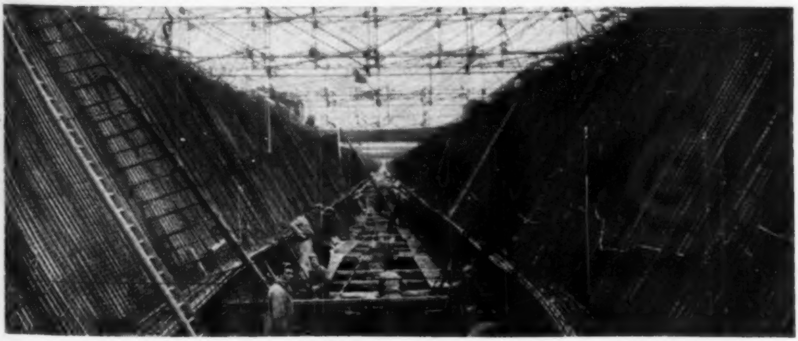


**Fig. 6.—Belgian Civil Engineering Building during Construction.**

The weight of the cantilever is largely counterbalanced by the weight of a suspended hall, the floor of which forms an equilateral triangle of 85 ft. side. The floor consists of a slab 4·8 in. thick supported on six beams, which radiate from the central pillar to a bowstring girder at the rear of the hall. The arch of this girder is formed by the rearmost edge of the shell roof over the hall, which is 2·4 in. thick; its surface is that traced by a vertical parabolic generatrix following a directing curve approximating to a cubic parabola. The suspension rods of the girder are of aluminium, and are reinforced with mild steel plates to enable them to resist bending moments due to the wind. The arch loading is transmitted to the central pillar by two prestressed concrete members, the cross sections of which are 15 in. square and which contain 96 prestressing wires of 7 mm. diameter.

The whole structure is supported on piles; fourteen vertical piles and fourteen raking piles support the central pillar, and seven raking piles are provided under each side strut. The two side struts and the central pillar are also connected by a triangular reinforced concrete foundation beam.

*Fig.* 7 shows a model of a pavilion which was designed by M. Le Corbusier; the method of construction in precast prestressed concrete was suggested by Dr. H. C. Duyster of the Belgian concern Société des Travaux en Beton et Drageges. The structure will comprise some 2000 precast slabs measuring about 40 in. square by 2 in. thick, all of different shapes; they will be kept in position by prestressing wires of 0·28 in. diameter tensioned after the slabs are erected.

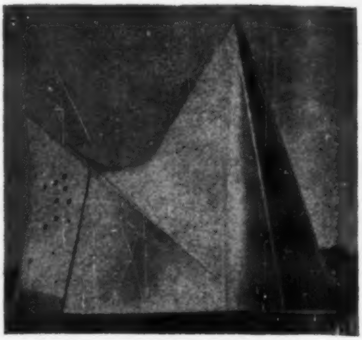


**Fig. 7.—Pavilion Designed by M. Le Corbusier.**

*Fig.* 8 shows a model of the French pavilion.

**Fig. 8.—The French Pavilion.**

# Prestressed Concrete with Pre-tensioned Steel.

## Loss of Prestress due to Elastic Shortening of the Concrete.

By PAUL W. ABELES, D.Sc., M.I.Struct.E.

It is well known that when the prestressing force is transferred from the anchorage to the concrete by bond (that is the pre-tensioning process), thereby producing compression in the concrete, the elongation of the steel due to tension is instantaneously reduced as a result of the elastic shortening of the concrete. For each tensioned steel member the amount of this reduction is determined from the condition that the strains in the steel and in the concrete adjacent to the steel must be equal, that is $\frac{p_i - p_t}{E_s} = \frac{f_{st}}{E_c}$, in which $p_i$ is the initial tensile stress in the steel, $p_t$ is the tensile stress in the steel at the time when the prestressing force is applied to the concrete (known as "at transfer"), and $f_{st}$ is the compressive stress at transfer in the concrete adjacent to the steel. This equation can be written in the form $Lp_e = p_i - p_t = m.f_{st}$, in which $Lp_e$ is the loss of prestress due to elastic shortening and $m$ is the modular ratio $\frac{E_s}{E_c}$. When the pre-tensioning process is used, it is usual for the compressive stress in the concrete to vary from about zero at the top to a maximum, $f_{1T}$, at the bottom. To obtain such a distribution of stress it is necessary to apply the prestressing force at the limit of the kern, or core, which is at a distance $e_k = \frac{r^2}{e_2}$ below the centroid; $r^2$ is the square of the radius of gyration $\left(= \frac{I}{A}\right)$ and $e_2$ is the distance of the top edge from the centroid. For a rectangular section $e_k = \frac{D}{6}$, as is indicated in *Fig.* 1.

The tensioned steel is often distributed over the entire section. This arrangement was favoured at a period when the design was based solely on working-load conditions and before it became general practice to consider also ultimate-load conditions. Nevertheless this kind of distribution of prestressing steel (*Fig.* 2) is still sometimes used. Some fifteen years ago Professor R. H. Evans and the writer suggested an arrangement in which most of the steel was placed near the tensile edge (1), and the remaining part near the top edge (2) (*Fig.* 3). This

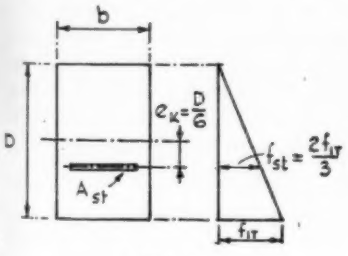



**Fig. 1.**

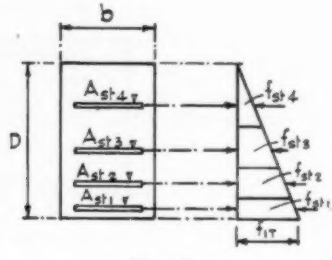



**Fig. 2.**

distribution improves the ultimate resistance, and greatly reduces the maximum width of the cracks, since all the tensile steel $A_{st}$ is closer to the tensile edge of the concrete.

In computing the losses of prestress due to elastic shortening, and also due to creep, it is often suggested that the prestressing steel may, with sufficient accuracy, be assumed to be concentrated at its centroid. This is satisfactory if the whole of the prestressing steel is placed in several closely-spaced layers in the tensile zone, but this kind of assessment leads to a gross underestimation of the losses if it is employed for arrangements such as those in *Figs.* 2 and 3.

The losses of prestress in the steel according to *Fig.* 1 ($Lp_e = p_i - p_t$) may be obtained either by direct computation or by trial and error. In the latter case the magnitude of the compressive stress $f_{1T}$ in the concrete must be assumed first, whereas in the former case it is assumed to be $f_{1T} = \dfrac{2A_s . p_t}{b.D} = 2p.p_t$, in which $p = \dfrac{A_s}{b.D}$ is the ratio of the cross-sectional areas of steel and concrete. As the stress at the limit of the kern is two-thirds of the maximum stress in the concrete, $p_i - p_t = \frac{4}{3}m.p.p_t$, and the reduction factor $R_t = \dfrac{p_t}{p_i} = \dfrac{3}{3 + 4m.p}$.

If, for example, $m = 5$ and $p = 0{\cdot}01$ (that is 1 per cent. of steel), then $R_t = \dfrac{3}{3{\cdot}20}$ $= 0{\cdot}9375$ ; the losses due to elastic shortening are $6\frac{1}{4}$ per cent. and if $p_i = 150{,}000$ lb. per square inch the maximum compressive stress $f_{1T}$ amounts to $2 \times 0{\cdot}9375 \times 0{\cdot}01 \times 150{,}000 = 2812$ lb. per square inch, which is close to the permissible upper limit. At the more suitable stress of $f_{1T} = 2500$ lb. per square inch the percentage is slightly less than 1.

If the loss is determined by trial and error it is first necessary to assume a value for this loss. For example, if a loss of 5 per cent. is assumed, the stress $f_{1T}$ amounts to $2 \times 0{\cdot}95 \times 0{\cdot}01 \times 150{,}000 = 2850$ lb. per square inch and the loss $Lp_e = 5 \times \frac{2}{3} \times 2830 = 9433$ lb. per square inch, giving $R_t = \dfrac{140{,}567}{150{,}000}$ $= 0{\cdot}9371$. It is clear, therefore, that even the first attempt leads to a sufficiently exact result of 6·7 per cent. instead of the assumed value of 5 per cent., and this can be verified by repeating the computation with the improved value.

The same problem will now be investigated using the arrangement given in

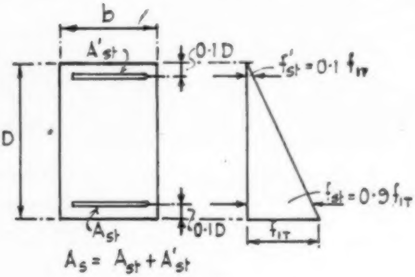



**Fig. 3.**

*Fig.* 3. It is assumed that the prestressing steel is placed at a distance of $0{\cdot}1D$ from the outer edges. From the two conditions

$$A_{st} + A'_{st} = A_s \quad \text{and} \quad (A_{st} + A'_{st})\frac{D}{6} = (A_{st} - A'_{st}) \times 0{\cdot}4D,$$

$$A_{st} = 0{\cdot}411A'_{st} = \frac{1}{1{\cdot}411}A_s = 0{\cdot}71A_s \quad \text{and} \quad A'_{st} = 0{\cdot}29A_s.$$

If the stress $f_{1T}$ in the concrete is assumed to be, for example, 2800 lb. per square inch, corresponding to an average reduction factor of 0·933 (= 2800 ÷ 3000), then the corresponding stresses in the concrete adjacent to $A_{st}$ and $A'_{st}$ are $f_{st} = 2520$ and $f'_{st} = 280$ lb. per square inch respectively, and the corresponding losses of the stress in the steel are given by $Lp_e = 5 \times 2520 = 12{,}600$ lb. per square inch (giving $p_t = 137{,}400$ lb. per square inch) and $Lp_e' = 5 \times 280 = 1400$ lb. per square inch (giving $p_t' = 148{,}600$ lb. per square inch). For sections such as that shown in *Fig.* 3 the effects of $A_{st}$ and $A'_{st}$ on the stress $f_{1T}$ at the bottom edge are expressed by the coefficients

$$k_1 = 1 + \frac{e_1 . e_s}{r^2} = 1 + 3 \times 0{\cdot}8 = 3{\cdot}4 \quad \text{and} \quad k_1' = 1 - 3 \times 0{\cdot}8 = -1{\cdot}4,$$

in which
$$\frac{e_1 . e_s}{r^2} = \frac{e_1 . e_s'}{r^2} = 3 \times \frac{e_s}{0{\cdot}5D},$$

as the square of the radius of gyration for a rectangle is $r^2 = \frac{D^2}{12}$, the eccentricities $e_s$ and $e_s'$ are $0{\cdot}4D$ and $e_1 = e_2 = 0{\cdot}5D$. It follows that

$$f_{1T} = (3{\cdot}4 \times 0{\cdot}71 \times 137{,}400 - 1{\cdot}4 \times 0{\cdot}29 \times 148{,}600)\ 0{\cdot}01$$
$$= (331{,}200 - 60{,}400)\ 0{\cdot}01 = 2708 \text{ lb. per square inch.}$$

Repeating the calculation with the improved value of $f_{1T}$, $f'_{st} = 271$ and $f_{st} = 2437$ lb. per square inch; $Lp_e = 5 \times 2437 = 12{,}185$ lb. per square inch; $p_t = 137{,}815$ lb. per square inch; $Lp_e' = 5 \times 271 = 1350$ lb. per square inch; $p_t' = 148{,}650$ lb. per square inch; and

$$s_{1T} = (3{\cdot}4 \times 0{\cdot}71 \times 137{,}815 - 1{\cdot}4 \times 0{\cdot}29 \times 148{,}650)\ 0{\cdot}01$$
$$= (332{,}200 - 60{,}400)\ 0{\cdot}01 = 2718 \text{ lb. per square inch.}$$

From this, $R_t = \frac{2718}{3000} = 0{\cdot}906$, and the reduction of the effective stress in the concrete at the bottom of the beam due to elastic shortening of the concrete is not 6·25 per cent. but 9·4 per cent., which is 50 per cent. greater than would have been obtained had it been assumed that the steel was concentrated at its centroid.

It is clear that the method of first assuming the loss and then correcting the assumed value is sufficiently accurate, but it is also possible to calculate the losses exactly, as is indicated in *Figs.* 4 and 5, in which the influences of $A_{st} = 0{\cdot}71A_s$ and $A'_{st} = 0{\cdot}29A_s$ are considered separately.

In *Fig.* 4 $\quad f_{1T} = 3{\cdot}4 \times 0{\cdot}71 \times 0{\cdot}01\ p_t = +0{\cdot}02414\ p_t$
and $\quad f_{2T} = -1{\cdot}4 \times 0{\cdot}71 \times 0{\cdot}01\ p_t = -0{\cdot}00994\ p_t.$
In *Fig.* 5 $\quad f_{1T} = -1{\cdot}4 \times 0{\cdot}29 \times 0{\cdot}01\ p_t = -0{\cdot}00416\ p_t$
and $\quad f_{2T} = +3{\cdot}4 \times 0{\cdot}71 \times 0{\cdot}01\ p_t = +0{\cdot}00986\ p_t.$

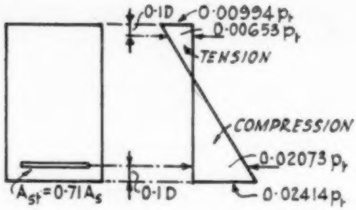



**Fig. 4.**

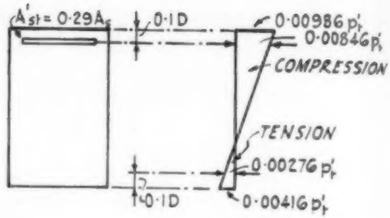



**Fig. 5.**

For the stress diagram shown in *Fig.* 4, the stresses in the concrete adjacent to the steel areas $A_{st}$ and $A'_{st}$ are

$$f_{st} = (0{\cdot}02414 - 0{\cdot}1 \times 0{\cdot}03408)\ p_t = 0{\cdot}02073\ p_t$$

and $$f'_{st} = (-\,0{\cdot}00994 + 0{\cdot}00341)\ p_t' = -\,0{\cdot}00653\ p_t'\,;$$

in *Fig.* 5 $$f_{st} = (-\,0{\cdot}00416 + 0{\cdot}00140)\ p_t = -\,0{\cdot}00276\ p_t$$

and $$f'_{st} = (0{\cdot}00986 - 0{\cdot}00140)\ p_t' = +\,0{\cdot}00846\ p_t'.$$

From these expressions the following equations can be obtained for the losses of prestress due to elastic shortening.

$$p_i - p_t = 0{\cdot}02073m\,.\,p_t - 0{\cdot}00276m\,.\,p_t' \quad . \quad . \quad . \quad (1)$$

$$p_i - p_t' = 0{\cdot}00846m\,.\,p_t' - 0{\cdot}00653m\,.\,p_t \quad . \quad . \quad . \quad (2)$$

Eliminating $p_i$, $p_t(1 + 0{\cdot}02726m) = p_t'(1 + 0{\cdot}01122m)$, and for $m = 5$,

$$p_t' = \frac{1 + 0{\cdot}02726m}{1 + 0{\cdot}01122m}\,.\,p_t = \frac{1{\cdot}1363}{1{\cdot}0561}p_t = 1{\cdot}076p_t.$$

Hence $$p_i - p_t = 0{\cdot}10365p_t - 1{\cdot}076 \times 0{\cdot}0138p_t\,;$$

$$p_t = \frac{p_i}{1{\cdot}0888} = 0{\cdot}92p_i\,; \text{ and } p_t' = 0{\cdot}99p_i.$$

Therefore

$$f_{1T} = (3{\cdot}4 \times 0{\cdot}71 \times 0{\cdot}92 - 1{\cdot}4 \times 0{\cdot}29 \times 0{\cdot}99) \times 0{\cdot}01 \times 150{,}000$$
$$= 2725 \text{ lb. per square inch,}$$

which is in good agreement with the approximate value of 2718 lb. per square inch previously obtained.

From a comparison of the results in *Figs.* 1 and 3 obtained by exact calculation and by trial and error it is apparent that a sufficiently exact assessment of the loss due to elastic shortening is obtained by using the first assumed value of $f_{1T}$. This can be improved by a repetition of the calculation. On the other hand, the assumption that all the steel may be considered to act at its centroid leads to grave errors, and this over-simplification should not be used. It may be said that the error in this case is not great if it is related to the total value of $f_{1T}$. For example, the difference between the stresses $f_{1T}$ at transfer calculated according to these two methods (*Figs.* 1 and 3) amounts to $2812 - 2718 = 94$ lb. per square inch, which, when related to 2718 lb. per square inch, is only $3\frac{1}{2}$ per cent. However, this value must be related to the losses due to elastic shortening obtained by the use of this over-simplification, in which case the error is 50 per

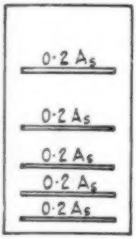



Fig. 6.

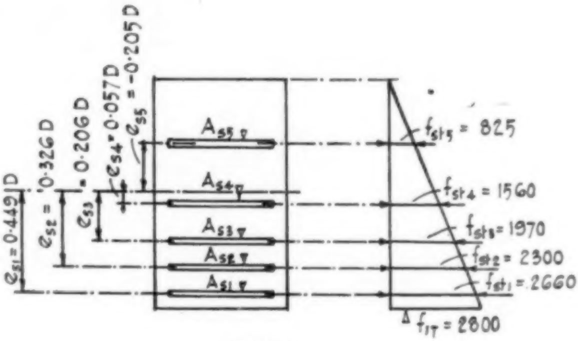



Fig. 7.

cent. It should be remembered that many assumptions are made in the calculation; for example, Young's modulus of elasticity of the concrete must be assessed and an error may occur in this assessment. Consequently, any possibility of increasing the unreliability of the calculation by making unnecessary over-simplifications should be avoided, even if the error amounts to much less than 50 per cent.

## Further Examples.

In the following, an example according to *Fig.* 2 is investigated. In *Fig.* 6 the area $A_s$ is divided into five equal parts; for the purpose of obtaining a triangular distribution of stress, these areas must be positioned as shown in *Fig.* 7. The suffix $n$ relates to the separate areas of reinforcement $A_{sn}$, in which $n$ varies between 1 and 5. For each of these centroids the corresponding value of $k_{1n}$ related to the bottom edge can be computed from $k_{1n} = 1 + 3\dfrac{e_{sn}}{0{\cdot}5D}$, in which the distances $e_{sn}$ from mid-depth downwards are assumed to be positive. The stress $f_{1T}$ is then obtained from the general formula

$$f_{1T} = \Sigma k_{1n}\left(\frac{A_{sn}}{A_s}\right)(p_i - m\,.f_{stn})p.$$

This is evaluated in *Table* 1, in which $p_i = 150{,}000$ lb. per square inch and $p = 0{\cdot}01$; $f_{1T}$ is assumed to be 2800 lb. per square inch. Multiplication by

TABLE I.

| $n$ | $\dfrac{e_{sn}}{D}$ | $k_{1n}$ | $\dfrac{f_{stn}}{f_{1T}}$ | $f_{stn}$ | $m.f_{stn}$ | $p_i - m.f_{stn}$ | $k_{1n}(p_i - m f_{stn}).p$ |
|---|---|---|---|---|---|---|---|
| 1 | +0·449 | +3·694 | 0·949 | 2660 | 13,300 | 136,700 | +5040 |
| 2 | +0·326 | +2·956 | 0·826 | 2300 | 11,500 | 138,500 | +4100 |
| 3 | +0·206 | +2·236 | 0·706 | 1970 | 9850 | 140,150 | +3130 |
| 4 | +0·057 | +1·342 | 0·557 | 1560 | 7800 | 142,200 | +1910 |
| 5 | −0·205 | −0·230 | 0·295 | 825 | 4125 | 145,875 | − 340 |
| | | | | | | | +13,840 |

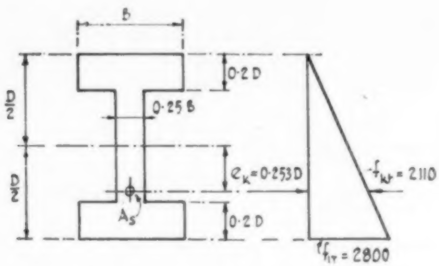



**Fig. 8.**

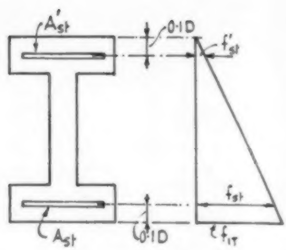



**Fig. 9.**

$\dfrac{A_{sn}}{A_s} = 0{\cdot}2$ gives $f_{1T} = 0{\cdot}2 \times 13{,}840 = 2768$ lb. per square inch. In this case the error is less than in the case of the examples shown in *Fig.* 3, but it amounts to $2812 - 2768 = 44$ lb. per square inch, which is 23·5 per cent. of the incorrect value of 188 lb. per square inch.

Finally, the difference between over-simplification and approximate assessment is shown for an I-section according to *Fig.* 8, of which the cross-sectional area is $A = (1 - 0{\cdot}75 \times 0{\cdot}6)\, B.D = 0{\cdot}55\, B.D$; the second moment of area is $I = (1 - 0{\cdot}75 \times 0{\cdot}6^3)\dfrac{B.D^3}{12} = 0{\cdot}838\dfrac{B.D^3}{12}$; the square of the radius of gyration is $r^2 = 1{\cdot}524\dfrac{D^2}{12}$; and the distance of the limit of the kern from the centroid is $e_k = \dfrac{1{\cdot}52D^2}{12.\dfrac{D}{2}} = 0{\cdot}253D$. Hence $\dfrac{f_{kt}}{f_{1T}} = 0{\cdot}753$. If $f_{1T}$ is again assumed to be 2800 lb. per square inch, $f_{kt} = 0{\cdot}753 \times 2800 = 2110$ lb. per square inch, $mf_{kt} = 10{,}550$ and $p_t = 139{,}450$ lb. per square inch, from which

$$f_{1T} = 2 \times 0{\cdot}01 \times 139{,}450 = 2789 \text{ lb. per square inch.}$$

On the other hand, as shown in *Fig.* 9, if $A_{st}$ and $A'_{st}$ are assumed to be at a distance $\dfrac{e_s}{D} = 0{\cdot}4$ then

$$k_1 = 1 + \frac{12 \times 0{\cdot}5 \times 0{\cdot}4}{1{\cdot}524} = 2{\cdot}578, \text{ whereas } k_1' = 1 - 1{\cdot}578 = -0{\cdot}578.$$

The corresponding values of prestress in the steel are

$$p_t = 150{,}000 - 5 \times 0{\cdot}9 \times 2800 = 150{,}000 - 12{,}600 = 137{,}400, \text{ and}$$
$$p_t' = 150{,}000 - 5 \times 0{\cdot}1 \times 2800 = 148{,}600 \text{ lb. per square inch}$$

and the required areas are obtained from $(A_{st} - A'_{st})0{\cdot}4D = (A_{st} + A'_{st})0{\cdot}253D$.

This gives $\quad A'_{st} = \dfrac{0{\cdot}147}{0{\cdot}653}A_{st} = 0{\cdot}225A_{st}$,

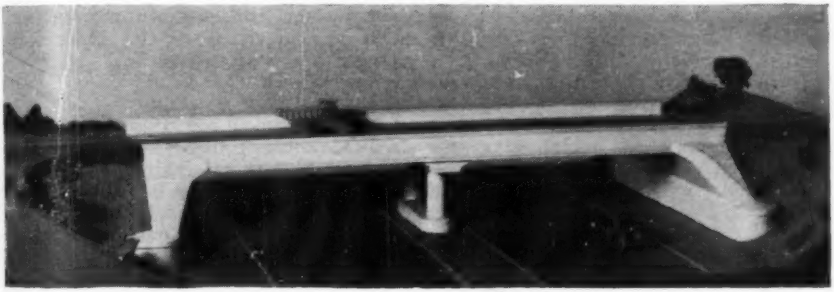
from which $\quad A_{st} = \dfrac{1}{1{\cdot}225}A_s = 0{\cdot}816A_s$ and $A'_{st} = 0{\cdot}184A_s$.

The stress $f_{1T}$ is then obtained from

$$f_{1T} = 2{\cdot}578 \times 0{\cdot}816 \times 137{,}400 \times 0{\cdot}01 - 0{\cdot}578 \times 0{\cdot}184 \times 148{,}600 \times 0{\cdot}01$$
$$= 2900 - 158 = 2742 \text{ lb. per square inch.}$$

Also in this case the difference is small, namely $2789 - 2742 = 47$ lb. per square inch. Related to the loss itself, however, computed by the first method as 211 lb. per square inch, it is 22 per cent.

## The London–Yorkshire Motorway.



WORK on the southern part of the London–Yorkshire motorway, which will extend from near St. Albans to near Rugby, will start within the next two months, and is expected to be completed in nineteen months. The contractors are Messrs. John Laing & Son, Ltd., and the cost will be about £15,000,000.

The work will include the construction of 53 miles of dual carriageways and more than 100 bridges. The road will comprise 4 inches of asphalt on 14 inches of water-bound Macadam. The bridges will be of plain and reinforced concrete; the illustration shows a model of a standard square over-bridge, comprising a reinforced concrete slab with central supports.

The work is under the direction of the Ministry of Transport and Civil Aviation, and the consulting engineers are Sir Owen Williams & Partners.

## Prestressed Concrete Development Group.

THE committee of the Prestressed Concrete Development Group for the year 1958 has been elected as follows.

Lt.-Col. G. W. Kirkland, M.B.E. (Chairman), Col. W. B. Sykes, C.B.E., T.D. (Vice-Chairman), Mr. J. Singleton-Green (immediate Past Chairman). Mr. H. G. Cousins, Mr. A. Goldstein, and Mr. A. J. Harris (engineers). Mr. G. M. Adie, Mr. E. G. Dean, and Mr. A. Kirkwood Dodds, M.C. (architects). Mr. R. T. Betts, M.B.E., Mr. E. W. H. Gifford, and Mr. R. F. T. Kingsbury (specialist engineers). Mr. J. W. A. Ager, Mr. J. M. Fisher, and Mr. A. Moller (contractors). Mr. J. Barratt, Mr. J. Beattie, and Mr. K. R. Danhof (products manufacturers). Mr. J. W. Gibson, Mr. K. G. Hann, and Mr. A. E. Osborne (manufacturers of prestressing steel). Mr. A. W. Hill, (chief technical officer). Mr. P. Gooding (secretary).

A Northern Ireland Regional Committee of the Group has been formed. Mr. S. Taggart, managing director of Messrs. Farrans, Ltd., of Dunmurry, is the chairman, Mr. V. Smyth, an architect, is the vice-chairman, and Mr. J. McClure, consulting engineer, the honorary secretary.

# Requirements for Fire Resistance.

WE have received the following from the Director of the Fire Research Station of the Department of Scientific and Industrial Research and Fire Offices' Committee.

I have read the Editorial Note in your issue of September, 1957, relating to requirements for resistance to fire of reinforced concrete elements, and since there seems to be some doubt as to the reconciliation of the recommendations in British Standard Code of Practice No. 114 (1957) with the notional periods attributed to these elements in the bye-laws of the various preceptive authorities, it may be helpful to explain that the modifications have an experimental basis and do not merely represent the opinions of this organisation.

The Model Byelaws of the Ministry of Housing and Local Government and the By-laws of the London County Council contain tables of common elements of construction and the fire resistance which may be attributed to them. These tables were based largely on those given in Post War Building Studies No. 20, Fire Grading of Buildings, the data for which were obtained from tests made at the Fire Research Station in accordance with the 1932 edition of British Standard No. 476.

In 1953 a revised edition of B.S. No. 476 was issued in which the following changes were made in the conditions for fire resistance testing. (1) The design load, instead of 1½ times design load as formerly, was required to be imposed during the test; (2) The application of a water-jet from a fire hose at the end of the heating period was no longer required. For load-bearing elements therefore the test under the conditions of the revised British Standard was much less severe than under the previous British Standard, and reinforced concrete elements, and particularly columns, would then be likely to show to greater advantage than other forms of construction. A limited amount of test data was available when C.P. No. 114 was being prepared on the effect of these changes in the test conditions on the performance of reinforced concrete elements, and these data were therefore included in the Code. Investigations now in hand will provide the means for revising some of the information in the Code which, through lack of test evidence, is in the same form as it appears in the Byelaws.

Although the tables of fire-resistance ratings in the Model Byelaws are derived from tests made in accordance with B.S. No. 476: *1932*, it is provided that a part of a building shall be deemed to have a given fire resistance if it has been shown by the appropriate test of B.S. No. 476: *1953* on a similar part to have that fire resistance (Model Byelaw 31). Consistency in fire-resistance requirements will be achieved when the Byelaws are revised, but until revision takes place it is desirable that the results of our work should be made available in order that the older forms of construction are not at a disadvantage compared with the newer forms. A British Standard Code of Practice is a suitable channel, in our opinion, for making results available.

In the cases where the Code refers to mesh reinforcement we are making available the results of tests, and use is made of data from the results of tests. A mesh reinforcement of relatively small-diameter wires embedded in concrete exposed to fire does not form a plane of weakness from which the concrete may spall, as occurs with normal reinforcement. Although separation may occur at the concrete/main steel interfaces the mesh will hold the cover in place without its value being impaired as an insulator to the reinforcement. This may not be true if the thickness of concrete over the mesh is large, but tests have shown that with concrete covers of up to 1½ in. over the main reinforcement a mesh wrapped round the bars can act as effectively as when it is located centrally in the concrete cover.

The Editorial Note mentions the recommendations in the Code relating to the use of limestone-aggregate concrete for columns, and it is pointed out that the by-laws do not recognise it for use in this way. It will be noted, however, that both the Model Byelaws and the London County Council By-laws list limestone as a Class 1 aggregate for concrete in walls, where it shows to advantage over Class 2 aggregates. Tests have shown that reinforced concrete columns made with limestone aggregate are superior in fire resistance to those made with Class 2 aggregates.

# Replacing a Viaduct by a Cellular Embankment.

A VIADUCT which carries the main railway line over the River Wheelock at Elton, Cheshire, is to be replaced by a cellular embankment. Subsidence caused by the pumping of brine has necessitated frequent lifting of the bridge and raising of the abutments and wing-walls and it was recently necessary to insert steel struts between the abutments (*Fig.* 1). The subsidence between 1892 and 1956 was 16 ft., and the present rate of settlement is 8 in. a year. It is thought that the cellular construction to be provided through the bridge opening will allow large subsidence to occur without further raising and strengthening of the existing abutments.

The new construction (*Fig.* 2) will consist of layers of reinforced concrete pipes, of 5 ft. external diameter, laid through the opening and curtailed in length as the work proceeds to follow the outline of the existing embankments. The honeycombed embankment so formed will rest on a reinforced concrete raft, and the pipes will be concreted together so that the raft and pipes will act as a single structure.

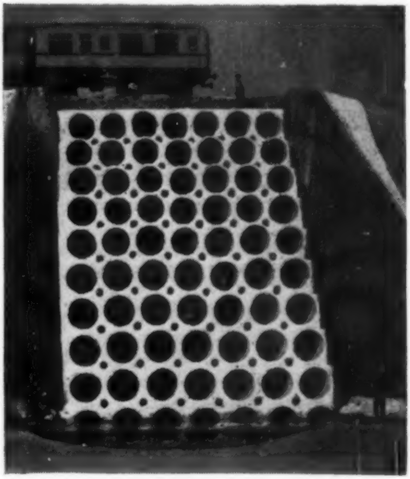


**Fig. 2.—Model of New Work.**

**Fig. 1.**

Boreholes indicated that the river-bed consists of a mixture of sand and silt to a depth of at least 15 ft. Investigation of the soil appeared to indicate that if the load transmitted to the ground in the early stages of the work does not exceed 1 ton per square foot the settlement will not exceed a few inches before the construction is completed. If greater settlement occurs, it is thought that the low ultimate bearing capacity of the ground, even after it is compacted by the weight of the structure, is not likely to allow large contact pressures to occur, a maximum of $2\frac{1}{4}$ tons per square foot being anticipated.

The structure is designed to resist lateral pressures exerted by the embankment, and the spaces between the pipes and the existing abutments will be filled with concrete. No work will be carried out on the abutments.

The sequence of construction will be as follows. (1) Sheet piling will be driven across the river at each side of the bridge, and the water will be pumped through the bridge opening through pipes placed higher than the bottom layer of concrete

pipes in the new construction. (2) The reinforced concrete raft will be placed, settlement being prevented by the sheet piling. (3) The first layer of concrete pipes will be concreted in position. (4) The sheet piling will be withdrawn and the river allowed to flow through the concrete pipes, the temporary piping and pumps being removed. (5) The layers of concrete pipes will be built up. The struts between the abutments will be removed as the pipes are built up to the underside of the struts. (6) The superstructure of the existing bridge will be removed and the railway track relaid.

The work is to be carried out by the London-Midland Region of British Railways.

## Resistance to Impact of Prestressed Beams.

In "Building Research, 1956" (H.M. Stationery Office. Price 5*s*. 6*d*.) it is stated that the experimental comparison of the behaviour of prestressed concrete beams and reinforced concrete beams under impact in the laboratory at the Building Research Station and in full-scale tests has now been completed. It was found that prestressed concrete behaves as an elastic material, with a high capacity for recovery after severe deformation, whereas reinforced concrete has little capacity for recovery but a much higher capacity for absorbing energy, as it sustains large permanent deformation before structural failure occurs. Transverse reinforcement is essential in both forms of construction to ensure that failure due to bending occurs without premature failure due to shearing, which severely reduces resistance to impact. The method or system used for prestressing did not appear to have an important influence on the resistance to impact of normally designed beams.

Several series of composite prestressed concrete beams designed to fail due to fracture of the steel have been tested under repeated loading. Each series was prestressed by a different form of wire, with a diameter of 0·2 in. Although the results show that the use of deformed or indented wires leads to reduced resistance to fatigue, the margin of security against failure due to fatigue remains adequate. In all the tests the loading could be repeated one million times from 0·5 times the design working load to at least 1·55 times the design working load, equivalent to a range of stress in the steel of from 58 to 74 tons per square inch.

## Hydrophobic Cement.

A Portland cement that is resistant to moisture for long periods, and which therefore can be stored without deteriorating in damp conditions, is now supplied under the name Pectacrete by the Rugby Portland Cement Co., Ltd.

In this journal for January 1952 it was reported that this type of cement was being produced in the U.S.S.R. In our number for December 1952 it was reported that a similar cement had been produced in 1950 by the A.S.P. Chemical Co., Ltd., of Gerrards Cross, and in this journal for March 1957 it was announced that this type of cement was being produced and sold under the name "Hydracrete" by the Cement Marketing Co., Ltd.

In our August 1953 number a report was given of some tests made at the Building Research Station which showed that the addition to cement clinker of oleic acid in proportions of up to 1 per cent. gave the desired result. The chemical forms a waterproof film that coats the grains of cement, and which is rubbed off when the cement is subjected to abrasion with the aggregates during the mixing process so that the setting and hardening properties of the cement are the same as in the case of untreated cement.

## Gatwick Airport.

We regret a mistake in the January number of this journal. The foundations and prestressed concrete at Gatwick Airport are by Sir Alfred McAlpine & Son, Ltd., and the superstructure is by the Turriff Construction Corporation Ltd., and not as stated on page 60.

# A Factory with a Precast Shell Roof.

A CIGARETTE factory now being built near Ballymena, Northern Ireland, will have two production halls of identical construction placed parallel to each other, with annexes on three sides. They will be linked at one end by a canteen, a welfare department, and administration offices. All the structures are of reinforced concrete and consist largely of units precast on the site.

## The Production Halls.

The requirements for the production halls were that there should be close control of the temperature of the air and the humidity, absolute cleanliness, and no internal obstructions. Single-story buildings with north-light shell roofs were considered to meet these requirements most effectively. One of the halls is 600 ft. long and the other 900 ft. long; both are 105 ft. wide. There are no internal columns in either hall (*Fig.* 1). In each hall hollow precast main beams, which also serve as air-conditioning ducts and contain other services, span the full width; they have a curved outline, the radius of the soffit being 302 ft. 9 in. *Figs.* 2, 3, and 4 show sections through the hall. The beams are at 30 ft. centres and are simply supported at each end on hollow columns, 2 ft. wide and 14 ft. 10 in. high, cast in place. The columns on one side of the building include rocker-bearings to allow for expansion of the beams. Services are brought to the main beam, through the hollow columns, from a reinforced concrete tunnel about 8 ft. wide by 6 ft. deep which extends the full length of each building under a corridor in the annexes. There are also service trenches at intervals across the factory, with connections to the tunnel.

Each main beam is 9 ft. deep and 4 ft. wide and consists of seven precast hollow units, with walls 4 in. thick and about 12 ft. 8 in. long, and six intermediate hollow stiffening units about 2 ft. long with walls 9 in. thick. To form the beams the units are connected by fourteen prestressing cables each comprising twelve wires, using the Gifford-Udall-CCL system. The units are precast in two channel-shaped sections each 2 ft. deep; the joint at the top of the beam is mortared during erection, and the halves are temporarily bolted together across the top and bottom until the prestress has been applied.

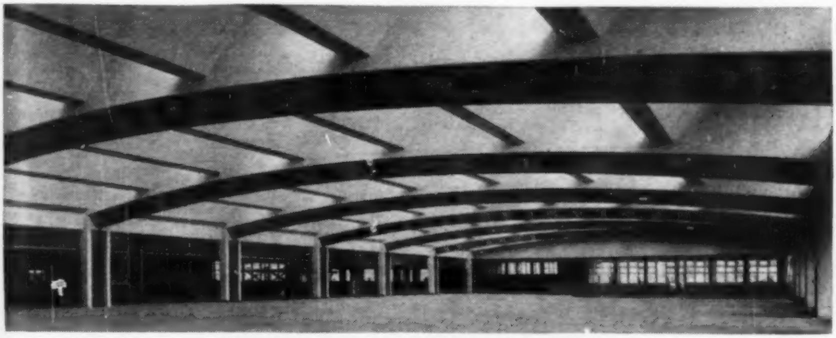


Fig. 1.—One of the Production Halls.

Seven precast north-light shell units (*Fig.* 6), placed side by side, span between the main beams. The rise of the shell is contained within the depth of the beams. Each of these units is about 15 ft. wide and 3 in. thick, and weighs about 9 tons. The valley-beams are precast separately and are supported on the stiffening units that form parts of the main beams. By placing the stiffening units within the walls of the reinforcement connecting the main beams with the shells and valley-beams it was possible to reduce the thickness of the walls of the beams.

All the precast units are erected by means of a derrick crane of 15-tons capacity (*Fig.* 7). While a main beam is being erected and prestressed the crane is used to place the shell units and valley-beams between the two beams previously

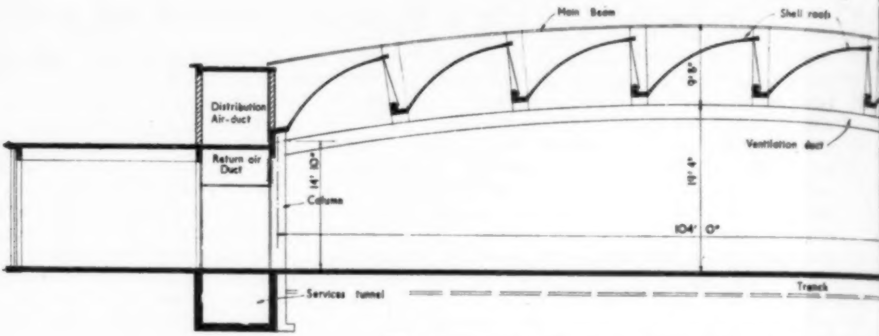



**Fig. 2.—Cross Section through Production Hall.**

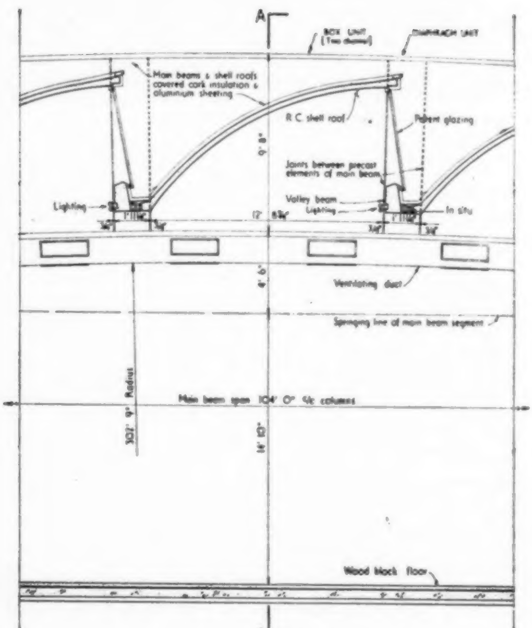



**Fig. 3.—Part Cross Section.**

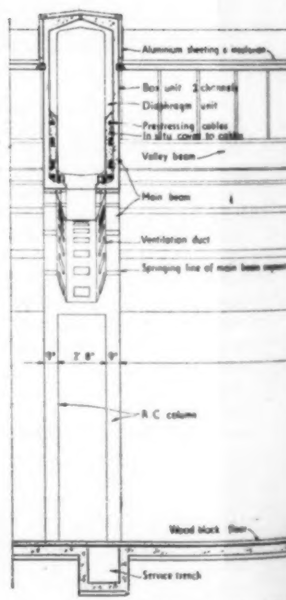
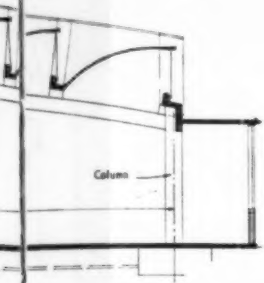


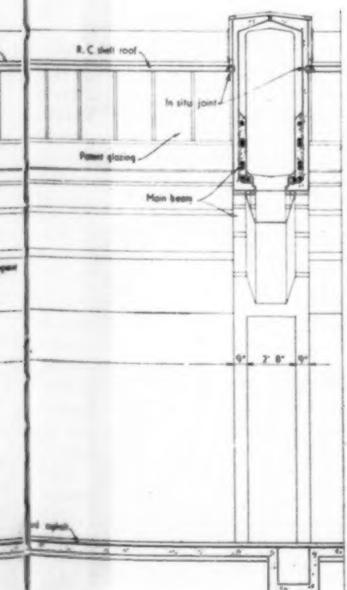

**Fig. 4.—Part Lo**




ll.

erected. A steel frame is attached to the shells during lifting (*Fig.* 5). Joints between units are formed with concrete cast in place. The average time for completion of one bay of the building is nine days. The beam units are lifted into position and supported on a timber gantry until they have been mortared together and prestressed. Two days after the mortar is applied the prestressing cables are threaded through the beams and the prestress is applied. The cables are contained inside the beam, seven against each side. In the beams the cables are initially exposed, except that in the stiffening units they are threaded through ducts in the side walls. The end-blocks containing the anchors are essentially similar to the stiffening units. After stressing, the cables arranged against the walls of the beam are covered with concrete and grout is used where the cables pass through holes in the sides of the stiffening units. The provision of complete cover to the cables is important since the air in the beams will be continually in an almost saturated condition.

Transverse expansion joints are provided at intervals of about 200 ft. Each joint divides the main beams longitudinally and the hollow columns vertically. To ensure that the beams will be completely stable until the roof is placed, a wooden filler is used in place of a mortared butt joint. A similar filler is also inserted during the precasting of the stiffening units. After erection of the shells on both sides of a beam, the joint is formed by knocking out the filler and cutting the reinforcement between the two parts of the stiffening units.




**Lo[ngitu]dinal Section A—A.**

### The Annexes.

Precast columns and beams form the framework of the annexes. The beams span 22 ft. 6 in. and 26 ft. 3 in.; the columns are 9 in. square and 14 in. square. At one end of each production hall a large annexe, four bays wide, is provided for preliminary manufacturing processes. At intervals along each hall the annexes on the south side are extended to provide accommodation at first-floor level for the air-conditioning plants; there are six of these for one factory and four for the other.

### Other Buildings.

The two-story office building curves away from one of the production halls to follow the river and is linked to the other hall at its mid-point by a single-story cloakroom building. The canteen is at the end of the main wing and is connected with it by an entrance hall tower, which is partially clad with precast facing-slabs.

The framework of the offices and cloakrooms is precast, with floor and roof slabs, $5\frac{1}{2}$ in. thick, cast in place. The beams span 18 ft. and 22 ft., and the columns are 12 in. by 9 in. in cross section. The canteen is a single-story building with a shell roof, and will accommodate 850 people. It is entirely precast, and the frames which support the roof are prestressed. Each frame is precast in three parts, namely two upright parts with cantilevered haunches 10 ft. long, and a central part 40 ft. long. The prestress is applied by means of two 12-wire cables.

**Fig. 5.—Erecting a Roof Slab.**

**Fig. 6.—Top of Roof.**

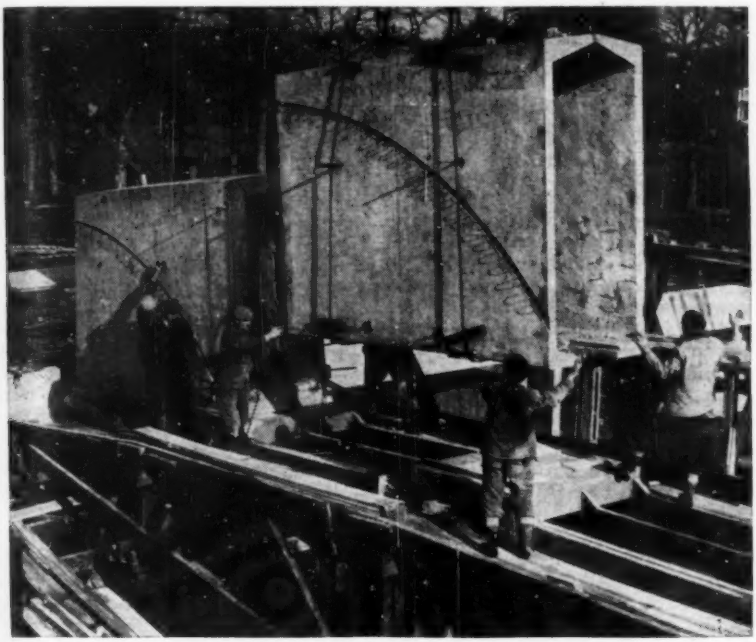


**Fig. 7.—Erecting a Hollow Beam.**

All the structural members are precast in a yard, about 100 ft. by 300 ft., which is served by a derrick-crane of 15 tons capacity. The shuttering is generally of timber, with plywood linings for exposed surfaces. About thirty uses of the shuttering are obtained before reconditioning becomes necessary. A 1 : 4½ concrete is used, and the shuttering is stripped after three days. Average compressive strengths of 8000 lb. per square inch at 28 days are obtained from the test cubes.

The consulting engineers are Sir Alexander Gibb & Partners, and the contractors are Sir Alfred McAlpine & Son, Ltd.

## New Road from Birmingham to Bristol.

TENDERS have been accepted for the first two sections, totalling ten miles, of the proposed new road between the Midlands and South Wales. Contract No. 1 is for a length of seven miles including a by-pass 1½ miles long at Ross, and has been awarded to Tarmac Civil Engineering, Ltd., and contract No. 4, for three miles, including a bridge, to Messrs. A. E. Farr, Ltd. The road will have two 24-ft. carriageways.

At Queenhill (in contract No. 4) a bridge 2466 ft. long will be built over the river Severn. It will have a central span of nearly 240 ft. and two side spans of 130 ft. each of steel construction with a reinforced concrete deck. The approaches will have nine spans of 82 ft. on the west bank and fifteen on the east bank, all of reinforced concrete. The bridge was designed by Sir Alexander Gibb & Partners.

Where the by-pass crosses the river Wye at Bridstow there will be a prestressed concrete bridge 353 ft. long with a 203-ft. span and two 75-ft. approach spans. The engineers are Messrs. Scott & Wilson, Kirkpatrick & Partners.

# Book Reviews.

**"Statically-Indeterminate Structures."** By R. Gartner. (London: Concrete Publications, Ltd. Price 18*s.*; by post 19*s.* $4 in Canada and U.S.A.)

THE third edition of this book has been extensively revised. The chapters dealing with the linear analysis of structures—the importance of which is becoming more widely appreciated—are retained, and new chapters describing the choice of unknown forces and moments and the analysis of structures by the plastic-hinge method are included. Much additional information has been included in the chapters dealing with the distribution of moments and the analysis of curved beams.

**"Reinforced Concrete."** By J. C. Maxwell-Cook. 380 pages. (London: English Universities Press, Ltd. Price 32*s.* 6*d.*)

THE fundamentals of reinforced concrete design are explained in this well-illustrated book written for students. It contains seventeen chapters, the last of which gives 110 examination questions set by professional institutions. All chapters include worked numerical examples and exercises. The chapters deal with the general principles of design, foundations, retaining walls, tanks, and prestressed concrete. Appendixes give design data, a specification, a bar-bending schedule, typical reinforcement details, and a bibliography.

The book is a useful one, but it could be improved. For example, in several cases bars are chosen of $\frac{1}{16}$-in. sizes from $\frac{3}{16}$-in. to $1\frac{3}{16}$-in. Variations by $\frac{1}{8}$ in. are better in practice and avoid using callipers. Anomalies in the assessment of stresses due to "punching shear" at 100 lb. to 200 lb. per square inch are frequent; the Code of Practice for Reinforced Concrete (No. 114) does not recommend such stresses. The stresses adopted for the design of tanks are higher than those recommended in the Institution of Civil Engineers' Code on Liquid Retaining Structures. The chapter on prestressed concrete is good. The bibliography might have included B.S. No. 1478 : 1948 on bar-bending measurements, Code of Practice No. 4 (1954) on Foundations, and the I.C.E. Code on Liquid-retaining Structures. The appendix on reinforcement details could be improved.—R. P. M.

**"Concrete Technology."** By F. S. Fulton. (Johannesburg: The Concrete Association. 486 pages. Price 20*s.*)

THIS is a compendium of every aspect of the making and placing of concrete and the materials used. The information has been gathered from many sources, and useful bibliographies are given at the end of each chapter. The author is the Director of the South African Concrete Association, and although the book deals specifically with practice in South Africa the information is of general application and of value to engineers in other countries.

**"Kempe's Engineers' Year-Book."** [London: Morgan Brothers (Publishers) Ltd. Price (2 volumes), 82*s.* 6*d.*]

THIS monumental work of reference, now in its 63rd edition, needs no introduction to our readers. New features include more information on new processes of welding and on water engineering; details of new blasting processes; new tables relating to wire ropes; revised text on tides and movement of water in relation to docks and harbours; new information on heating, ventilating, and air-conditioning; and a revision of the chapter on reinforced concrete in accordance with B.S. Code of Practice No. 114.

**"Building Technicians' Pocket Diary",** 1958. (London: Association of Building Technicians. Price 6*s.* 2*d.* by post.)

THE 37th edition includes maps, calendars, ruled graph paper, memoranda pages, and 160 pages of data on measuring, planning, structural engineering, and the building trades, and a summary of legislation relating to building.

## A Tall Building in Manchester.

THE building illustrated on page 52 of our January number was designed by the Chief Architect's Department of the Ministry of Works. The consulting engineers are Messrs. R. Travers Morgan & Partners, acting under the direction of the Chief Structural Engineer of the Ministry of Works.

# Prestressed Bridges in Ceylon.

**By K. H. BEST, B.Eng., A.M.I.C.E., M.I.Struct.E.**

THE scarcity of steel, the high cost of transport, and import duties have resulted in the increased use of prestressed concrete for highway bridges in Ceylon during recent years. Until 1955, when the construction of a prestressed highway bridge 310 ft. long was commenced by local contractors under the supervision of British consulting engineers, structural steel and reinforced concrete were used, the latter being preferred as most of the materials were available locally. Since 1948 more attention has been given to the improvement of the highway system, and many bridges have been constructed. Those described, all of which are in prestressed concrete, were designed and supervised by Messrs. Husband & Co., consulting engineers.

## Weragantota Bridge.

This bridge crosses the Mahaweli Ganga, the largest river in Ceylon, and is in flat jungle country immediately after the river leaves the central hills. The normal width of the river is about 800 ft., but the maximum flow when the river is in flood is considerably greater and the water level has a range of about 30 ft.

The bridge (*Fig.* 1) has six spans each of 130 ft. The deck comprises ten precast prestressed beams, each 5 ft. 9 in. deep, with a top flange 29 in. wide, a web 6 in. thick, and a lower flange 20 in. wide. Each beam weighs about 57 tons, and is prestressed by eleven post-tensioned Freyssinet cables, ten containing twelve wires and one containing eight wires.

The end-blocks, which contain the anchor cones, are precast and are reinforced with mild steel. In order to reduce the quantity of high-tensile steel required, some of the cables are bent up and anchored in the top flange of the beam at positions where recesses were provided and cones are cast in. A typical arrangement of the cables is shown in elevation in *Fig.* 2, and a cross section through the deck is shown in *Fig.* 3.

The ten beams are placed side by side on reinforced precast bearings; these are alternately fixed and free at the piers. The expansion bearings comprise short concrete rockers with cylindrical surfaces; a sheet of rolled lead is provided between surfaces in contact. Transverse prestress is applied to the beams after erection.

The piers have a face of 1 : 2 : 4 reinforced concrete 9 in. thick surrounding a core of plain 1 : 3 : 6 concrete. The piers are connected to the capping slabs over the foundations by means of steel rails (*Fig.* 4). Three cylinders are provided below each main pier. These are 12 ft. in diameter and are sunk to rock through the river bed, which is composed mainly of silt and fine sand; they are plugged with 1 : 2 : 4 concrete and filled with 1 : 3 : 6 concrete.

The contractors were Messrs. Gammon (India) Private, Ltd., who elected to use the Freyssinet system of prestressing. The contract drawings and documents also included alternative methods of erection. The first provided for casting the main beams in separate short units, which would be erected and prestressed together on a temporary girder. The other method suggested, which was chosen by the contractor, is outlined in *Fig.* 5. An aluminium truss 145 ft. long with a triangular cross section was used. The bottom boom of the girder (*Figs.* 6 and 7) comprised a Y-shaped extruded section, and the beams were suspended from this girder on pairs of carriages having roller-wheels bearing on the bottom flange. After the completion of one span of the deck, the girder was released from its supports at the piers and, after extra panels were added at the back, was connected to a travelling mast to which guy-ropes were fitted to enable the nose of the truss to be cantilevered. The whole assembly was then moved forward across the deck and placed between the next piers.

The proportions of the concrete for the main beams were generally 1 : 1·5 : 3·6 by volume, or 1 : 5 by weight. Of the coarse aggregate 30 per cent. was between 1 in. and ¾ in. and 70 per cent. was smaller than ¾ in., of which 10 per cent. was smaller than ⅜ in. The sand had a fineness modulus of 2·7 and 40 per cent. of voids. The sand comprised about 33 per cent. by weight of the total aggregate. Bulking of the aggregate varied from 27 per cent. to 33 per cent., with an average of about 28 per cent.

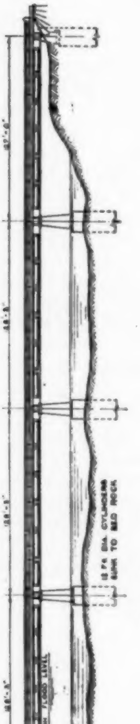


Fig. 1.

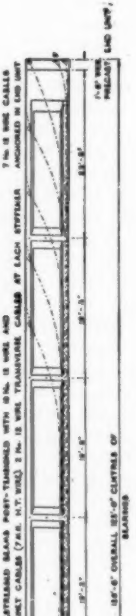


Fig. 2.

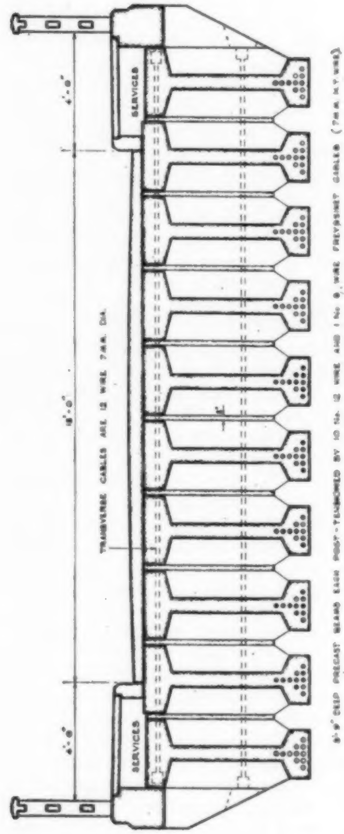


Fig. 3.
Weragantota Bridge.

The water-cement ratio was generally 0·4. A richer concrete was used in the bottom flange, the proportions being 1·2 : 1·5 : 3·6, with a slightly higher water content to produce greater workability. The average results of the cube tests are shown in *Fig.* 8. It was considered advisable to use rich concrete in the bottom flange of these relatively large beams. This permitted the use of a higher water-cement ratio in order to improve workability, and the analysis of the cube tests given in *Fig.* 8 indicates the importance of the water content; despite the richer concrete the average strength of the cubes made from the concrete for the lower flanges was slightly lower than elsewhere.

Considerable attention was given to the method of casting the beams so as to prevent the formation of cracks at the re-entrant angle between the top flange and the web, which is a trouble that has been reported in tropical conditions.*

*W. E. Dean. Prestressed Concrete—Difficulties Overcome in Florida Bridge Practice. "Civil Engineering" (U.S.A.), June 1957.

Before commencing the full-scale production of the beams, trial lengths were cast in order to determine the best method. The beams were not cast in one continuous operation, vertical construction joints being provided at third-points. Powerful external vibrators were clamped at intervals to the outside of the steel shutters, which were of robust construction.

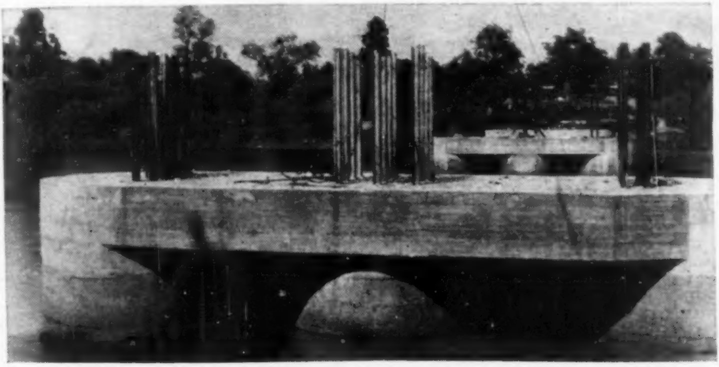


**Fig. 4.—Foundations for River Piers.**

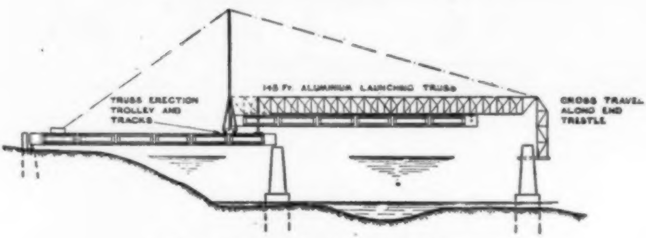



**Fig. 5.—Method of Erecting Beams.**

The stresses in the beams at the time of erection and under full Ministry of Transport loading were:

Full dead load: At the top, +972 lb. per square inch; at the bottom, +1290 lb. per square inch.

Full Ministry of Transport load: At the top, +1970 lb. per square inch; at the bottom, −10 lb. per square inch.

Before the beams were prestressed, consideration was given to the effects of friction and also to the most suitable sequence for tensioning the main cables, and trials were made in advance. Calculations showed that the effects of the progressive elastic shortening of the concrete as the cables were tensioned

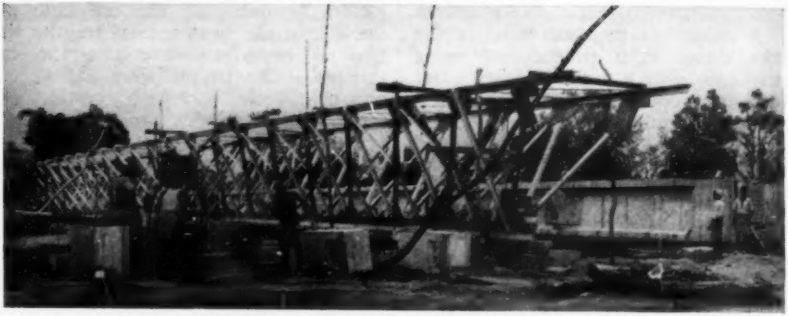


Fig. 6.—Beams at Casting Yard.

Fig. 7.—Launching Girders for Weragantota Bridge.

| CUBE STRENGTHS AT 28 DAYS LBS. / SQ. IN. | PERCENTAGE OF TOTAL TESTS | | AVERAGE STRENGTH OF GROUP | |
|---|---|---|---|---|
| | A | B | A | B |
| 6000 – 7000 | 27 | 28 | 6450 | 6530 |
| 7000 – 7500 | 38 | 32·5 | 7150 | 7250 |
| ABOVE 7500 | 35 | 39·5 | 7790 | 7950 |

RESULTS A ARE TAKEN FROM BOTTOM FLANGE OF BEAMS

RESULTS B ARE TAKEN FROM ELSEWHERE IN BEAMS

Fig. 8.—Results of Crushing Tests.

Fig. 9.

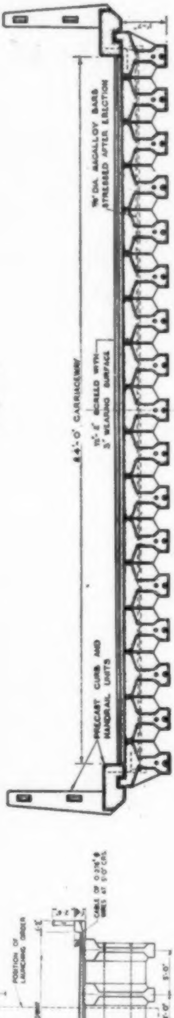


Fig. 11.

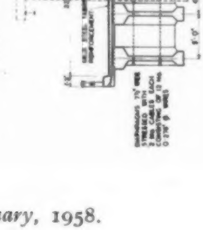


Fig. 10.

required an initial jacking force of nearly 80 tons per square inch in some of the cables. The initial stress required at mid-span was calculated to be about 52 tons per square inch, allowing for the effects of the wires acting as reinforcement in the grouted cables. Estimates were made of the losses due to friction in the ducts, and the sequence of tensioning the cables was chosen so as to limit the stresses applied by the jacks to a maximum of 75 tons per square inch. After the trials it appeared that the actual frictional effects were less than those calculated ; this may be due to the fact that for these cables a lubricating solution was used. There was close agreement between the results obtained and the data issued to the contractors.

Reinforcement was provided in both flanges of the beams to resist any tendency to lateral bending during erection.

The use of a row of relatively large precast beams placed side by side is unusual for bridges with spans of 130 ft., and the decision to adopt this design was based on several factors. The construction depth was required to be as small as possible as the approach roads were relatively expensive ; a reduction in the number of beams would also have increased considerably the weights to be lifted, and this was the first time a scheme of this type and magnitude had been proposed in Ceylon. The use of large erection girders is common in India, and *Fig.* 9 indicates a design prepared by Messrs. Husband & Co. for a bridge over the river Udjh in Kashmir State. This has a length of 1155 ft., and appreciable economy is obtained by introducing continuity over the penultimate supports and by reducing the number of main beams to four. The cross section in *Fig.* 10 shows a proposal to erect two main beams together on each side of a central launching girder. In this case the launching equipment is heavy but is more readily available, and could also be used for a bridge of similar type some twenty miles away.

The engineers were required to train junior engineers seconded from the Public Works Department, and emphasis was therefore placed on the maintenance of proper records and the need for close collaboration between the engineers and the contractors on the site.

## Bridges at Gal Oya.

During the last few years a large land development scheme, including several bridges, has been in progress in the Gal Oya valley, in the Eastern Province, and consideration was given to the most suitable design, particular attention being paid to the possibility of standardisation and the establishment of a central factory for the production of precast units.

Throughout the valley sound rock is present generally at a shallow depth, although in the area of the main channel of the Gal Oya delta there is up to 50 ft. of varying grades of sand and alluvial deposits before bedrock is reached. A balance had therefore to be determined between the conditions over most of the area, which favoured short spans, and those where it might be advantageous to reduce foundation work by providing long spans. After making comparative designs and estimates, with particular reference to the number of bridges required, a standard span of 35 ft. was adopted, using precast prestressed beams of standard section (*Fig.* 11), which also provided a basic unit for some bridges of less than 35 ft. span.

The cross section of the deck of the standard 35 ft. span is shown in *Fig.* 11; the beams were placed side by side and prestressed transversely. Since the height of the decks above the level of the river-bed did not vary greatly in this area, it was also possible to standardise the foundations and piers. Where deep foundations were necessary a standard precast concrete cylinder of 5 ft. diameter, suitable for factory production, was designed. These cylinders were 4 ft. long, with ogee joints, and sunk in sections by open dredging to the depths necessary to give adequate protection against scour.

The contractors for this work were the Equipment & Construction Co., Ltd., who preferred to use the Lee-McCall system; consideration had been given to the possible establishment of a factory using the long-line method of prestressing, but the standard precast beam was redesigned to use Macalloy bars. Several thousands of these bars were placed and tensioned without any fractures or failures.

Close collaboration was maintained between the engineers and the contractors, whose responsibilities included the establishment of a factory for making the units in an isolated jungle area, using unskilled labour. *Fig.* 12 shows part of the factory, with curing by sprinklers in progress.

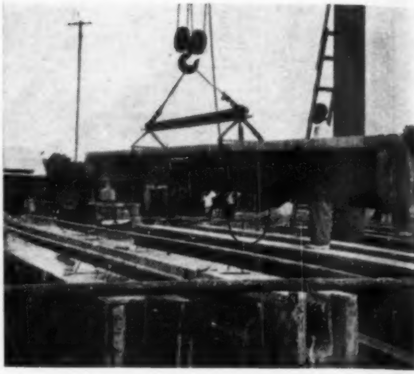


**Fig. 12.—Casting the Beams.**

The quality of the concrete was carefully controlled and comprehensive records were kept. The resident engineer kept a log giving detailed information on the test cube results, including the position in the work and dates of casting, for each beam. Analysis of the total results shows that at 28 days 16 per cent. had a compressive strength of over 8000 lb. per square inch, 27 per cent. between 7000 and 8000 lb., 54 per cent. between 6000 and 7000 lb., and 3 per cent. below 6000 lb. per square inch.

In the early stages of the work the nearest cube-testing machine was in Colombo, some 300 miles away. Cubes were taken by lorry over rough roads to the railhead and thence by rail to Colombo, a journey which took several days, and several cubes were damaged during transit. Bending tests were therefore made on the site on plain concrete beams 4 in. square by 16 in. long in accordance with B.S. Code of Practice No. 114. The moduli of rupture so obtained were correlated by means of Feret's expression with careful cube tests made at corresponding ages, and this was found to be particularly suitable for obtaining early estimates of the strength of the concrete in the beams. A few beams for which the test-cube results were below that specified were test-loaded in the factory before despatch. All these beams were found to be elastic under a short-period loading equal to 1·4 times the working load; no

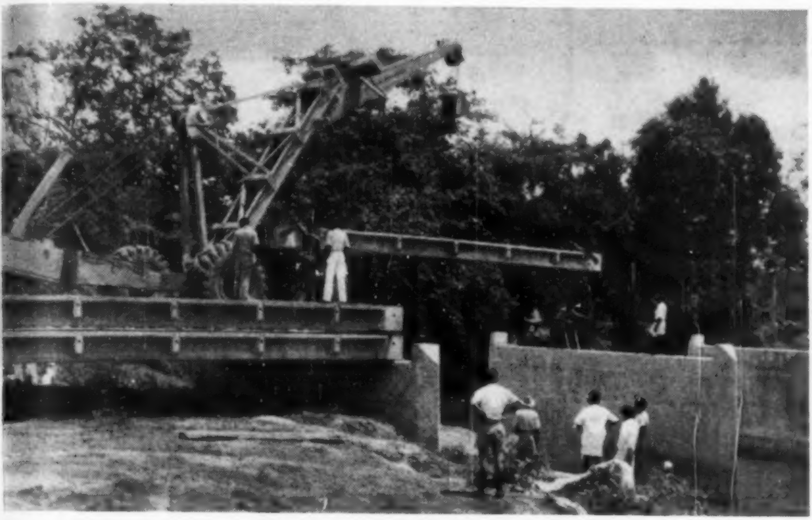


Fig. 13.—Erection of Beams.

cracks occurred, and they were accepted. Further tests to destruction were made on beams selected by the Gal Oya Development Board; the results indicated that the ultimate load was 3·2 times the working load.

The method of erection is shown in *Fig.* 13. Mobile cranes were generally used. Each beam weighs $3\frac{1}{4}$ tons, and a complete span comprising seventeen beams was easily erected in one day on a site several miles from the factory. The whole of the superstructure of each bridge, including the handrailing and parapet, was precast in the casting yard. *Figs.* 14 and 15 show two of the completed bridges.

The cost of these bridges was £4 per square foot where shallow foundations on bedrock were possible, and £6 per square foot where cylinders were sunk to depths between 20 ft. and 30 ft.

## Nanu Oya Bridge.

This bridge is of composite construction. The deck (*Fig.* 16) consists of a slab, 8 in. thick, of reinforced concrete cast in place and seven beams at 5 ft. centres. The main beams were cast on the shore and prestressed sufficiently to carry their own weight and the weight of the deck and transverse stiffeners. At

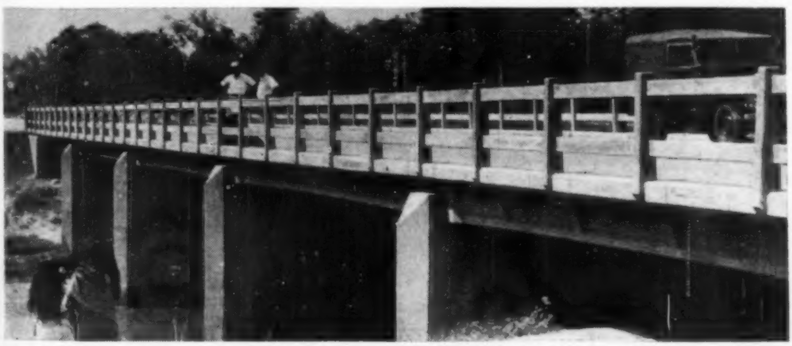


Fig. 14.—Gal Oya Bridge.

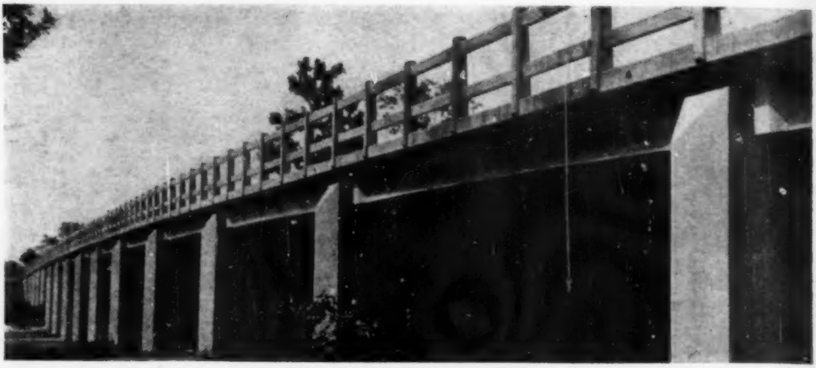


Fig. 15.—A Standard Bridge.

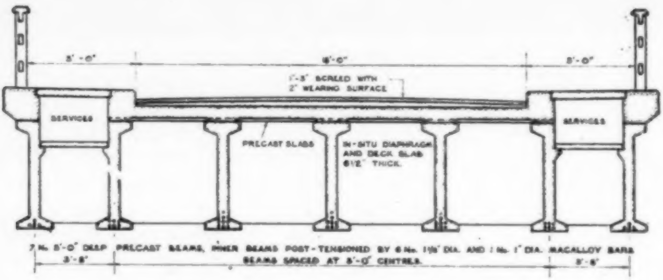



Fig. 16.—Nanu Oya Bridge.

| STAGE OF CONSTRUCTION | STRESS CONDITIONS | STRESSES IN LBS/SQ.IN. | | | |
|---|---|---|---|---|---|
| | | SLAB | | BEAMS | |
| | | TOP | BOTTOM | TOP | BOTTOM |
| BEAMS IN CASTING YARD | INITIAL PRESTRESS AND SELF WT. | | | − 204 | + 2304 |
| BEAMS ERECTED | AFTER LOSSES | | | − 164 | + 2150 |
| DECK CONCRETE POURED | UNDER WEIGHT OF WET CONCRETE | | | + 591 | + 1290 |
| DECK CONCRETE HARDENED | SHRINKAGE STRESSES ACTING | − 124 | − 164 | + 903 | + 1140 |
| REMAINING CABLES STRESSED | 2ND PRESTRESS | − 198 | − 186 | + 893 | + 1616 |
| SCREED AND SURFACING LAID | LOSSES AFTER PRESTRESS AND FURTHER D.L. | − 57 | − 90 | + 978 | + 1288 |
| FULL TRAFFIC ON BRIDGE | M. O. T. LIVE LOAD | + 693 | + 442 | + 1465 | − 192 |

Fig. 17.

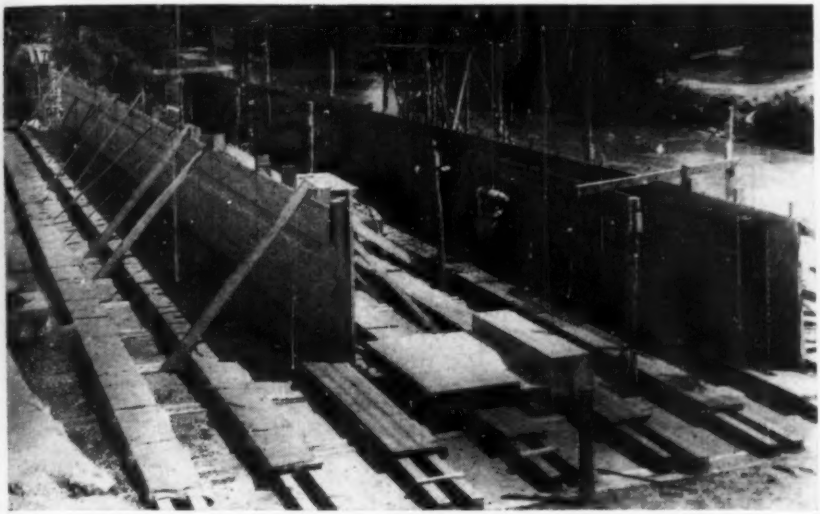


**Fig. 18.—Shuttering for Main Beams.**

this stage some of the bars in the main beams, which projected through the top flanges, were not tensioned. The beams were then erected and the deck slab was cast. After the concrete had hardened the remaining bars were tensioned, thereby prestressing the full composite section. Corrugations were provided in the top flanges of the precast beams, together with mild steel stirrups, to give the necessary resistance to shear. When analysing this section it was necessary to allow for the differential shrinkage between the precast beams and the slab. *Fig.* 17 indicates the estimated conditions of stress at the critical positions in the section at various stages of erection and prestressing, and *Fig.* 18 shows the shuttering for the beams in course of erection on the river bank.



**TENDERS.**

### Courses in Concrete.

PROSPECTUSES are now available of the training courses organised by the Cement and Concrete Association during the year 1958, starting on April 28. They include courses for engineers on the production of high-strength concrete, on the construction of concrete and soil-cement roads and runways, and on prestressed concrete; courses for supervisors, foremen, and clerks of works on making and consolidating high-quality concrete, on prestressed concrete, and on concrete and soil-cement roads; a course for architects, including lectures on farm buildings; a course for builders on methods of concrete construction; a course on precast concrete; and a course for lecturers. Each course is of $4\frac{1}{2}$ days' duration, and the fees do not exceed £5 including accommodation at a hostel. Full details are available from the Association at 52 Grosvenor Gardens, London, S.W.1.

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## *FIFTY YEARS AGO.*

From "CONCRETE AND CONSTRUCTIONAL ENGINEERING", JANUARY-FEBRUARY, 1908.

THE REBUILDING OF SAN FRANCISCO.—The excellent results shown by concrete and reinforced concrete in the earthquake and conflagration of San Francisco has necessarily brought about a considerable demand for these materials for the reconstruction of that city. Some of the reasons why it has not been possible to use reinforced concrete to the extent of the demand are given in the article,* but the primary reason is not sufficiently indicated. This reason was the unscrupulous methods of the brick and terra-cotta interests (both masters and men), which were employed to thwart the use of reinforced concrete in a manner characteristic of the very worst sides of American business and municipal life. Everything that could be done by bribery, corruption, threats, and riot was done in the interests of the brick and terra-cotta industry. That reinforced concrete should have successfully survived these attacks speaks well for its inherent good qualities and economy. Reinforced concrete is being used in over 100 important buildings in San Francisco, apart from its use on civil engineering works, and we may safely prophesy its almost universal use now that its application has become established in San Francisco, and the advantages are realised.

A CONCRETE FLY-WHEEL.—At a recent meeting of the Transvaal Institute of Mining Engineers a paper was read by Mr. W. D. Younger describing a novel use of concrete in a fly-wheel designed by Mr. D. Leitch, M.I.C.E. In the course of the discussion, Mr. A. Wallace pointed out that spur wheels, filled in with concrete, had been running for two and a half years at the Robinson Deep Mine.

* Printed in the same number.